EARTHQUAKES AND BUILDING CONSTRUCTION

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A Review of
Authoritative Engineering Data
and Records of
Experience

PRICE TWO DOLLARS

FOREWORD

The effect of earthquakes on various types of structures has been widely discussed in California in the last three years. In so far as such discussion has resulted in developing accurate data on which to base proper design and construction, it should serve a very useful purpose, and is to be commended. Unfortunately, however, much of it has consisted of the expression of individual opinions lacking adequate foundation on comprehensive engineering facts and principles. Experience has demonstrated that the earthquake is a phenomenon which must be taken into account in planning construction work; and has demonstrated, no less, that by taking it into account in an intelligent manner its destructive forces may be materially combatted, and the risk of damage from this source greatly lessened.

Manufacturers of burnt clay products in California have recognized the importance of this subject, but until recently have not carried out any thorough investigation of it. It has been known that well-built brick structures have withstood successfully earthquake shocks in every recent catastrophe of this nature. On the other hand numerous other structures have been demolished; unfortunately it is the buildings which fall, and not those which remain standing, which form the spectacular features of an earthquake and therefore attract public attention. Hence popular opinion is apt to be formed upon the failures rather than the successes in any type of structure, and without any analysis as to the true cause of the damage or its extent relative to the total amount of building.

This situation has led the Clay Products Institute of California to undertake extended research into this subject, accumulating data on the behavior of various building materials in recent earthquakes, as well as securing from engineers who have specialized in studying this problem, a careful technical analysis of the fundamental engineering principles involved.

The work of Bergstrom, Brunnier, Butts, Davis, Dewell, Hadley, Hill, Humphrey, Imamura, Jeffers, Jordan, MacElwane, Milne, Naito, Omori, Soulé, Townley, Willis and others has been carefully studied and drawn upon freely; formal discussions by Davis, Butts and Jeffers are embodied in this presentation.

In addition, many others throughout California have discussed the questions raised freely and fully and their ideas have contributed in an important way to the study.

Out of the data so obtained, the present report has been prepared which it is hoped may clarify the subject, and set forth in an authoritative manner both the merits and limitations of burnt clay products in building construction under earthquake conditions.

In preparing this report it has obviously been impossible to discuss adequately the properties of clay products without some comparisons with other materials. In doing this there is no desire to disparage any type of construction, but since in such matters as earthquake insurance the question becomes one of relative suitability of materials and types of construction, it is impossible to determine the proper rating of one class without a knowledge of the behavior of the others.

The present report was planned and edited by Seward C. Simons, Secretary-Manager of the Clay Products Institute of California during the time of its preparation, and by Norman W. Kelch, Chief Engineer of the Institute during the same period and subsequently Secretary-Manager. Mr. Kelch is a certified architect of long experience, an associate member of the American Society of Civil Engineers and his specialized knowledge of the intricate structural problems involved has contributed to the work in a most useful way. It has been no small task to select and arrange from the enormous amount of material the necessary data for an accurate, comprehensive and lucid report. It is not intended to be a complete treatise on either earthquakes or building construction, but a straightforward and authoritative discussion of the essential factors entering into building construction in earthquake regions.

The directors of the Clay Products Institute of California place this little work before the public confident that it will prove of material value to the California building industry and others allied with it.

ROBERT LINTON, *President,*
Clay Products Institute of California

CONTENTS

Earthquake Hazards

It does not require the aid of the seismologist to point out that earthquakes have occurred on the Pacific Coast during the last 150 years with considerable frequency. In spite of the fact that probably the most intense disturbance of this character such as the Mississippi Valley earthquake of 1811, or the Charleston earthquake of 1886 have occurred at far distant parts of the country and that other shocks are frequently experienced in the eastern part of the country as, for example, on the day that this chapter is being written, September 10, 1928, reports indicate a somewhat alarming tremor in Cleveland, Ohio, it is nevertheless true that if we may judge from experience, these earth movements are more frequent on the Pacific Coast than in many other sections of the country.

We disclaim any attempt in this discussion to enter the realm of the seismologist. We shall not attempt explanation as to the causes of earthquakes, nor discuss their history or character except as these may have a very direct bearing upon the design and construction of buildings.

It seems inherent in the nature of man to be terrified over demonstrations of natural phenomena unless they become frequent enough or well enough understood to take them out of the realm of mystery. The ancients were appalled at the eclipses of the sun, yet an eclipse of itself is not nearly as startling as a sunset, except that we have learned by repetition of the latter that it bodes no ill. The first experience of an earthquake is always terrifying, but when you talk to a person who has lived long in an earthquake zone, you find that much of this fear has vanished.

Because of the startling effect referred to, the records of the past visitations of such shocks to California are frequently more hysterical than historical. Such accounts as that given by the Portolá expedition of the earthquake of 1769 are of little value, and not until comparatively recent years has the science of seismology developed adequately to permit any accurate record as to the intensity of the earthquakes, while even the attempts to recount the damage in such shocks as that of 1857, are widely at variance.

Not only is it difficult to obtain any accurate information on the extent of the earlier earthquakes, but except in the most carefully written technical studies, it is hard to find uncolored record even of the most recent shake. Walter L. Huber, a distinguished consulting engineer of San Francisco, who has given a great deal of attention to damage from earthquakes in various parts of the country in an address delivered at the meeting of the Northern California Mortgage Bankers Association, August 11, 1927, said:

"To even those of us who have more or less constantly borne in mind for years the requirements of construction to resist earthquake shocks, the small percentage of damage done due solely to earthquake, in even major quakes, is interesting. It is not surprising that practically all earthquake reports are to some extent exaggerated. This is often wholly unintentional. The tendency is, however, to describe, or to photograph, the abnormal which, in these instances, is the building or structure which has failed or has suffered perceptibly. Its neighbor, next door, which may have been built in accordance with sound engineering principles and thereby escaped practically unharmed, gives the same appearance as it did last week or even last year and is, therefore, not the subject of much interest at the moment, particularly to the lay observer or photographer. Thus a record is left which is more or less complete so far as damage is concerned but very incomplete in so far as well built structures which were unharmed, or practically so, is concerned."

And in referring particularly to the San Francisco earthquake he added:

"It is interesting to observe tall brick chimneys, such as that of Spring Valley Water Company's Clarendon Heights pumping station, which was unharmed, and of even more interest is the stack of Black Point pumping station—veteran of the earthquakes of 1868 and 1906—still unharmed. All of the evidence points to the fact that the percentage of damage by earthquake as distinguished from fire damage to sound value, for the city as a whole, was small. This percentage was estimated at the time by two eminent engineers to be but 5%. While this percentage is of interest in connection with rates, it should not afford a false sense of security to the owner whose building may have been designed and built in violation of all principles of sound engineering. Such building may be one of the few which will completely collapse in the next earthquake."

If the unexplained statement is made that California is an earthquake country, and that history and an examination of present geological conditions indicates that there is every likelihood of a continued recurrence of earth shocks, the timorous resident and the eastern insurance executive or lender of building funds, might well be alarmed. If we add to this, however, the fact that we have been having these earthquakes since recorded history began; that with the exception of the losses due to the San Francisco fire indirectly caused

by the earthquake, the total damage over 150 years has been less than that of the Florida hurricane of September 1926; only a fraction of the losses occurring in the Mississippi flood of 1927; that the Santa Barbara earthquake caused less destruction of property than the Kansas City cyclone of 1925, the prospect does not seem so terrifying. Nor do the careful seismologists themselves regard the subject as darkly as has been generally assumed. Dr. S. D. Townley, one of the founders and for many years secretary of the Seismological Society of America, has written as follows:

"Investigation has shown that, except when a building was located right on a fault or very near a fault line, *none of the earthquakes of the past 150 years in California have been strong enough to seriously damage a well constructed building.* In the San Francisco earthquake, the steel and concrete class A structures were not seriously damaged and neither were well constructed brick and frame buildings."

Dr. James B. MacElwane, president of the Seismological Society of America, formerly for many years affiliated with the Department of Seismology, University of California and now Dean of the Graduate School of the University of St. Louis, in addressing a meeting at Pasadena, California, August 16, 1928, stated that there is no structural strain on the Pacific Coast at the present time and the danger of repetition of the 1906 earthquake is very slight. "Damages from earthquakes can be minimized and localized if we treat the problem as we treat the study of the weather," he declared.

This last statement is one which might well form the keynote of our entire presentation. We recognize that consideration should be given in the design and construction of buildings to earthquake forces just as consideration should be given to wind, rain, heat and cold and gravity. This subject should be considered in the same natural way in which the dangers from these forces are met. No good builder would locate a structure on a quicksand except from necessity, in which case he would take special precautions. Similarly a due regard for earthquake hazard would avoid construction near a recognized fault line. When the Scriptures tell of the man who built his house upon a rock they were referring to the rains, floods and winds, but as Palestine is an earthquake country the suggestion was especially sound. It is merely an example of providing against natural hazards. Likewise in the structure itself, there are certain elements of design which if duly regarded will greatly enhance the resistance of the building.

John R. Freeman, one of the best known engineers in the United States, who is also president of the Manufacturers Mutual Fire Insurance Company, Providence, R. I., writes to Clay Products Institute of California under date of March 28, 1929.

"I have for many years past been convinced that the earthquake damage has been grossly overestimated.

"I am personally very familiar with conditions around California, having done much engineering work there, having also been on the original commission to report on the Los Angeles aqueduct."

It will be found that in what we generally know as "good construction" there is in most cases a margin of safety which will be adequate to withstand earthquake stresses. It will also be found that there are dangers in some types of design which can readily be avoided, and at such comparatively small additional cost as to be negligible in view of the practical assurance of being earthquake proof. To discuss these elements and the extent to which buildings now found in California possess them as demonstrated by engineering analysis and the records of former severe earthquakes, and also to point out how it is possible to make our structures more earthquake resistant, will be the object of our later chapters.

CHAPTER II.

The Nature of Earthquake Forces

By WENDELL M. BUTTS
Consulting Structural Engineer

[NOTE: In order to form an adequate background for the discussion of the effect of earthquakes on building construction, it seems necessary to outline briefly the character and nature of the earthquake forces themselves. To present this discussion in an authoritative manner, we have asked Wendell M. Butts, consulting structural engineer of San Diego, to analyze the subject. Mr. Butts besides being past president of the San Diego Section of the American Society of Civil Engineers, is a member of the Seismological Society of America, was a member of the Engineering Committee of the Japanese Imperial Reconstruction Board, 1923-24, and is at this time a member of the Earthquake Investigation Committee of the Pacific Coast Building Officials Conference and a member of the California Standard Building Code Committee of the California Development Association. His many years of residence in the Orient gave him extensive contact with the distinguished seismologists of Japan who have done so much to clarify our notion of these disturbances.—*Editor.*]

Definition

Earthquakes are vibrations of the earth's crust produced by the sudden rupturing of the strata from great pressure. But to distinguish between earthquakes and other movements of the lithosphere, we may cite the technical definitions of Milne, viz:

"Earthquakes are sudden, violent movements of the crust. Tremors are minute quivers of the surface. Pulsations are gentle movements of long period. Oscillations are undulations of the crust, of great amplitude and very long period of great geological importance, but not sensible to any but the most precise measurements."

Some of these definitions have been questioned by modern writers, but they serve our present purpose to define earthquakes as the more violent movements to which the surface of the globe is addicted.

Waves

Now many forces are transmitted by wave motion in some medium. The force of the waves of the wind is observed in water waves. Sound is dissipated in the common acoustic waves. Light is transmitted by waves of very high velocity. The electric disturbances of a broadcasting station widens its effects through undulations of the electrical media and earthquakes widen their effect with phenomena similar to these wave motions.

As long as the impulse is relatively weak the wind on the water, the sound in the air, the beam of light and the radio waves and the earthquake vibrations are hardly noticeable. But, when the wind on the surface is violent and the waves are dissipated in shallow water, the fury of the storm may cause great havoc. The acoustic waves may be those of beautiful music or the clatter of a boiler factory. Light waves may bring to us the vision of a picture or the glare of blinding headlights. The electrical disturbances may be those of our radio or the deathdealing impulses of the little known lethal rays from a similar source. Likewise, the quivers of the earth's crust may pass unnoticed or they may bring in their wake the terrible destruction of an earthquake disaster.

Robert Mallet, an Englishman, after a profound study of the great earthquake and conflagration at Naples in 1857, which took a toll of 60,000 lives and destroyed Naples and several of the towns round about, and an investigation of the production and propagation of various waves, concluded as follows:

"An earthquake is the transit of waves of elastic compression in any direction through the crust of the earth from the center of impulse. It may be attended by sound or tidal waves, dependent upon the impulse and the circumstances of position as to sea and land."

This was the beginning of "Analytical Seismology." Seismology, or the study of earthquakes, up to this time had been mainly concerned with descriptions of various convulsions and usually assigned the origin to a displeased Deity and described the damage as a just punishment of a wicked people.

Causes

Earthquakes then, are sudden, violent movements of the earth's crust and the vibrations set up by a series of waves spreading from the origin. They may be due to one or a combination of causes. In some localities students ascribe the outbreaks to the seepage of water through the crust to a hot interior, creating superheated steam, demanding an outlet. This theory is held by those who have noticed that quakes are frequent where shores slope steeply to great ocean depths; where the crust is thinnest and the pressure greatest. Many Japanese earthquakes are supposed to be from this cause; the Tuscarora Deep, East of Japan, being the origin of frequent disturbances.

Other authorities blame volcanic activity for most shocks. Still other seismologists have discovered that

earthquakes sometimes result from the collapse of submarine or subterranean caverns formed by the dissolution of salt, lime or sulphur or the removal of mineral or oil deposits. Other disturbances are attributed to the movements of great land masses, induced by the cooling of the interior, or the shifting of the equilibrium of the land and water areas by rivers transporting enormous loads of soil from the continents to the sea.

Whatever the causes, it is the series of vibrations, radiating in all directions, which we call "earthquakes" and all forces incident to these convulsions, Mallet found, were transmitted by wave motion.

The Recording Seismograph

After this fundamental enunciation, there was little development until Milne, a Scotchman, Professor of Geology and Mining at the Japanese Imperial University, invented the recording seismograph, in 1883. At the time of his appointment to the chair of Geology at Tokio, he was already a man of considerable attainments as an explorer, mining engineer and geologist. As an indication of the energy and application characteristic of the man, it is interesting to recall that he spent a year tramping across Siberia to his new post, gathering data for memoirs on its geological characteristics. He was a veritable Darwin in his eager grasping of opportunity to study Nature's handiwork.

Seismoscopes had been in use in Europe for many years and the Chinese had used such instruments for a thousand years before Milne's attention was called to the need of an instrument to record these visitations, by a heavy earthquake the morning after his arrival in Tokio.

It is obvious that human observations are unreliable under conditions incident to an earthquake and although it is possible to calculate approximately, earthquake intensities from overturned objects found displaced after a convulsion, it is very necessary before these computations can be of any great value that many characteristics be known of the overturned object and its setting. Some mechanical means of measuring the intensity of the earthquake through a determination of its amplitude and the time required for a complete vibration, was essential. Seismoscopes were of many types; determining usually, the direction of the earthquake vibrations, but without mechanical means of leaving a definite written record. Seismographs, as the name indicates, are mechanisms for leaving a written record, that might be examined, after panic incident to the earthquake had subsided.

Earthquakes, Vibrations Simple Harmonic Motion

To an associate, Professor West, is accorded the distinction of having discovered that earthquake vibrations are *simple harmonic waves*, from a study of paths inscribed on seismograms. The definition of "simple harmonic motion" is "a linear movement in which the vibrating point has acceleration toward the center of the path proportional to its distance from the center." (Duff) The paths are sine curves.

Such a motion is the free swing of a pendulum. Acceleration has been defined as the change in velocity per unit of time. Velocity is usually expressed in feet or millimetres per second, and the second is the unit of time in which the speed is assumed to change. Acceleration is therefore expressed as a certain number of feet or millimetres per second per second.

The action of acceleration may be illustrated by the motion of a car. As long as the speed is constant, the acceleration is zero and there is no tendency for the occupants to be thrust forward or forced backward into the cushions. When the speed of the car is increasing there is an additional increment of speed per second of travel. This is positive acceleration and the passengers sink deeper into the cushions. When the brakes are suddenly applied, there is a decrease in velocity per unit of time and this is called negative acceleration, the occupants being thrown forward in their seats. It is this change in velocity, or the acceleration of the vibrations, which imparts to buildings the force of the earthquake.

This discovery of the nature of earthquake vibrations tied this motion and the forces set up to a known natural law, viz: Newton's Second Law of Gravitation. This provided a method of computing the effect of an earthquake on a body, which had hitherto been assumed as of infinite force.

Extensive experiments by Milne proved that all earthquake waves are simple harmonic, as are those produced in the earth's crust by dropping heavy weights or exploding charges of dynamite on the ground some little distance from the seismograph. Milne remarks in his celebrated book, "Earthquakes," that he had an abundance of material with which to work, for each week at least one of his stations was subjected to a severe disturbance.

Types of Waves

Through the use of these seismographs, thousands of records of the various types of vibrations were obtained and the investigators finally classified them into three primary waves, caused directly by the impulse, and one secondary disturbance, the result of the primary undulations. Different types of waves do not start from the origin, but varying speeds of transmission of the components filter the undulations, expanding in three dimensions, until the effect on a structure at the surface is of waves arriving separately.

Distortion of Waves

From the origin, which may be any shape, the vibrations expand in widening volumes and during their passage through a diversity of materials and through bodies of water, marshes, mountains and valleys, they are somewhat distorted. The famous wire models made by Dr. Sekiya, Professor of Seismology at Tokyo, show the paths of an earth particle under the influence of some earthquakes of which com-

plete records were obtainable. But seismograms also prove that these waves are many miles in length and the irregularities are therefore not appreciable on a building site.

Depth of Origin

Mallet computed the depth of the origin at Naples, in 1857, at seven or eight miles. Major Dutton estimated the depth of the earthquake at Charleston in 1886, as eleven to twelve miles and Bailey Willis estimates that the origin of the Santa Barbara earthquake of 1925, was ten or twelve miles below the surface. Daly says, "the most trustworthy observations indicate no depths of movement greater than twenty-five miles, the average being not far from half of this distance." This is important as indicating that there is considerable distance for the uniforming of the shocks before they reach the surface.

Length of Earthquake Waves

Various authorities have measured earthquake waves and have found lengths for the primary vibrations varying from ten to three hundred miles, with an average of approximately fifty miles. These lengths preclude the possibility of one end of a structure being affected by the crest of a wave, while the other rested in the trough thus causing varying intensities on different parts of the foundation.

Primary Waves

Of the primary vibrations we have the "up-and-down" or *vertical waves,* the *direct horizontal oscillations,* which tend to overturn in a vertical plane to-and-from a point on the surface over the origin and the *transverse horizontal movements* which also cause motion in a vertical plane but at right angles to the direct horizontal tremors.

Speed of Transmission

Again it must be emphasized that the vibrations of each series of waves affect the structure at different times, depending upon the separation produced by their differences in speeds of transmission and the distance to the origin. All start at the same time, but the *vertical* waves travel faster than the others and reach the particular site first. The next set of waves to arrive are the *direct horizontal,* and then the transverse horizontal. Usually these waves are over-lapping.

Vertical Waves

From the center of the disturbance down in the earth, elastic waves of compression move directly to the surface, striking buildings from beneath a considerable blow. Because persons are more sensitive to vertical than horizontal shock, the strength of these forces has been exaggerated and to them has been attributed much damage which they do not cause. Because the vertical vibrations present a rough relationship to the strength of the more destructive horizontal forces to follow, persons who have become inured to the panic usually attendant upon earthquake shocks, can tell how strong the ensuing disturbance will be upon the arrival of the first vertical jolts.

Direct Horizontal Waves

Next to arrive are the direct horizontal vibrations. These are the ones which cause much havoc because of their intensity and the fact that engineers and architects in the preparation of ordinary building designs are concerned largely with vertical and not such heavy horizontal forces.

Transverse Waves

The transverse tremors are felt last, usually after the other vibrations have ceased and always after the others have dropped below their peaks. These waves are due to the passage of impulses through solids and in this respect earthquake waves differ from sound or water waves.

Surface Waves

The primary vibrations set up disturbances in bodies of water, and soft and especially, wet ground, which are caused to heave and swell, like ocean breakers striking a coast, crashing against structures, sometimes with very destructive effects. These surface waves were agencies of great destruction in Charleston, and in Japan in 1923. In Charleston, reliable witnesses testified that the waves were eighteen inches high in soft ground and but a hundred odd feet from crest to crest. Instances are on record where the surface waves, in soft material, rose to fourteen feet in height and because they were comparatively short, a few hundred feet, cities have been destroyed by this phenomenon. The crushed sidewalk outside large buildings in Tokyo testify to the striking power of these surface movements.

Because the vertical and the transverse vibrations are relatively feeble, only the effects of the direct horizontal movements are observed in these secondary waves. For this reason, it is the usual practice to consider the horizontal effects as augmented by the surface waves.

Relative Intensities of the Waves

The approximate comparative values, from authoritative sources, for the vertical, the direct horizontal and the transverse waves are 2-6-1. For average intensities these values are more often 1-6-½, although 3-6-½ and 6-6-1 relationships seem to be indicated by some displaced objects in the Japanese earthquake of 1923.

Absolute Intensities of the Waves

To arrive at this we will resort to mathematical expression of these values. Now, the mathematical expression of Newton's Second Law of Gravitation is:

Force $=$ Mass $\times$ Acceleration, *the Mass being the property of the object acted upon and the acceleration representing the intensity of the earthquake:*

$$\text{Force} = \text{Mass} \times \text{Acceleration} = Ma$$

$$\text{Since Mass} = \frac{\text{Weight}}{\text{Acceleration of Gravity}}$$

$$\text{Force} = \frac{\text{Wt.} \times \text{Acceleration of the Earthquake}}{\text{Acceleration of Gravity}} = \frac{Wa}{g}$$

and, where Force of the Earthquake $= \text{``Q,''}$

$$Q = \frac{Wa}{g}$$

Thus, we have a formula to express the force induced by an earthquake when acting upon a body. As is evident, the force varies directly as the weight of the body and the ratio of the acceleration of the earthquake to that of gravity, $\dfrac{a}{g}$ sometimes written "K" and called the intensity of the earthquake waves.

The amplitude or the distance the body moves in either direction from rest, one-half of the total movement, is not in itself a measure of the intensity, except as the amplitude together with the period, or time consumed in a complete oscillation, may affect the acceleration. The relationship of the amplitude and the period are expressed in the following equation:

$$T = \sqrt{\frac{4\pi^2 A}{a}} \quad \therefore a = \frac{4\pi^2 A}{T^2} = \frac{39A}{T^2}$$

Where T stands for the period in seconds for one complete oscillation from A to C, A stands for amplitude in feet, half of the total motion, the distance from A to B, and, "a" represents the acceleration in feet per second per second, Fig. 1.
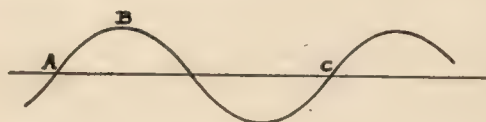

Figure 1

Thus, the acceleration, or the intensity of an earthquake varies directly as the amplitude of the movement and inversely as the square of the period. In comparing two quakes "X" and "Y", if "X" has a movement twice that of "Y", the period being the same, the intensity of "X" would be twice that of "Y". If, on the other hand, the amplitudes were equal and the period of "X" is one-half that of "Y", "X" would be four times as strong as "Y".

This explains why some earthquakes of small movement but great "rapidity" are more destructive than others of great amplitude but slower motion.

Expressed in terms of this absolute measure, (suggested by Milne and adopted by Omori and based on thousands of results from the computation of seismograph records and mathematical calculations of the force necessary to overturn objects such as gravestones, found displaced after an earthquake), the following are given as the approximate maximum components of record:

For the vertical acceleration—Five ft. per sec. per sec.

For the horizontal acceleration—Eighteen ft. per sec. per sec.

For the transverse acceleration—Three ft. per sec. per sec.

Effect of the Vertical Waves

Upon investigation of the effect of these separate components, we find that the greatest intensity for the vertical waves (five feet per sec. per sec.), when inserted in the formula:

$$Q = KW = \frac{aW}{g} = \frac{5}{32.16} W = .155 \ W \\ = 15\tfrac{1}{2}\% \text{ of the Weight of the body acted upon}$$

This will add to, or subtract from, the load actually carried by the vertical members of the structure about $15\tfrac{1}{2}\%$, a negligible amount, as columns are designed with factors of safety of four to six and footings are intended to have still higher margins for overload. It can then readily be seen that forces which add but 15% to the vertical loads, when buildings are designed to be safe against an increase in vertical load of 300 to 500%, may be neglected. The often expressed opinion that buildings are "lifted" and then "thrown down" by a disturbance is not confirmed by scientific facts or the observations of reliable witnesses.

Direct Horizontal Waves

These induce horizontal forces which are often dangerous unless buildings are especially designed to care for these conditions. In earthquakes on firm ground, the horizontal loads have a few times been in excess of 10% of the weight of the structure, and *where the sub-foundations were poor*, the intensity has reached a maximum of eighteen feet per second per second. By the method used in the examination of the vertical waves, they may be shown to set up horizontal loads of 56% of the weight of the structure. *These waves are certainly not to be neglected and we shall return to a detailed consideration of their effects later.*

Transverse Waves

Buildings are investigated for earthquakes at right angles to each of the faces. For this reason the transverse vibrations need not be considered as they are weaker than the direct-horizontal waves, also acting in a vertical plane, unless the two components acting together should produce torsion. The relative intensities of the two components are such that the effect

on the line of action of the direct horizontal force would cause the resultant to vary from the path of the greater force not more than ten degrees, Fig. 2.
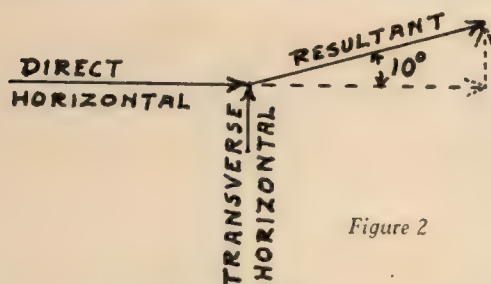

Figure 2

This may be neglected. There have been instances, however, where twin origins detonating one after another, as is not uncommon in the Tokyo-Yokohama District, have been reliably reported to have caused a whirling motion. This phenomenon is so rare, however, that it may be dismissed, except where the structure is a slender tower. Most observed tendencies to distort in a horizontal plane have been due to the shape of the structure, where the centers of the weights did not lie above the center of the plan of the buildings. No failure or damage has been attributed to torsion in any convulsion of which we have records, even in disturbances with twin origins, acting at right angles to one another.

Phenomena Other than Vibrations Induced by Earthquakes

Before we study the direct horizontal waves further, let us note some of the phenomena which often accompany seismic disturbances, viz: tidal waves, landslides and sudden depressions or elevations, sometimes of considerable magnitude.

In 1923, according to the reports of the Japanese Navy, several hundred square miles of Sagami Bay sank over one thousand feet. In the immediate neighborhood of Tokyo Bay thousands of acres of hydraulically filled or alluvial ground settled several feet. Slides have caused great loss of life and property in Japan, Switzerland and Italy. Fissures have opened and on a few occasions swallowed towns. Tidal waves have swamped hundreds of villages and drowned scores of thousands of the population in a single catastrophe.

Forces to be Considered

From the foregoing, it is evident that the forces of the earthquake against which provision must be made are the direct horizontal waves and that other forces may be neglected. *The intensity of the vibrations to reach the structure depends on the strength of the impulse, the formation on which the building rests and the distance of the origin from the site.*

Formation on Which Structure Rests

It is the universal observation that the damage is severest on loose or soft ground and least on solid rock. H. O. Wood presents data on the California Earthquake of 1906 to show what may be expected along this line:

	Acceleration	Ratios to Minimum
Dense Igneous rock, Potrero Hills	0.9 ft sec./sec.	1.0
Dense sandstone, Telegraph Hill	0.9 - 2.0	1.0 - 2.4
Gravel, sand and clay, more or less firmly compacted	2.0 - 3.5	2.4 - 4.4
Made land	3.5 - 9.6	4.4 - 11.6
Marshy ground	10 plus	12 or more

Observations in Japan indicate that a variation of more than three hundred per cent may be expected between the intensity of the quake on the more solid ground near the principal cities, and the lower alluvial soil, which unfortunately, is the sub-foundation of the business districts.

Distance from the Epicentral Tract

The distance of the site from a point, a line or an area over the origin, has a decided effect on the intensity of the disturbance to reach the place considered. Based on some data, it would appear that this varies approximately, inversely as the square of the distance when comparing localities. In other words, the relative intensities to be expected at two different places, distant five and ten miles from the epicenter are 1 and ¼ respectively.

The disturbed areas of quakes have varied greatly, dependent on the nature and the depth of the origin. The sources of the disturbances and whether they can be definitely expected to come from known fault lines are questions for seismologists to answer. That this question can be answered I believe possible. Bailey Willis says, "We are fully able, in most cases, to inform ourselves (of the location of faults in the vicinity of our cities) by making proper geologic surveys." "Faults and dislocations in the continuity of the strata which compose the earth's crust are the rule and not the exception, and imply the ceaseless action of powerful pressures and tensions, in suitable combination." (Knott). "There are in the neighborhood of 5000 perceptible earthquakes of the elastic rebound class each year. If each one were accompanied by a new break in the earth's crust, we would live on a very unstable old earth indeed. But that is not the case. Fresh shocks develop on old breaks and an old break may be regarded as not only having been, but certain in the future to be, the site of innumerable shocks." (Bailey Willis.) The moral is clear. *Avoid fault lines.* They are the source of most disturbances and, on this basis, the distance of the site from the fault becomes an important consideration.

Jordan says, "In a general way it seems to be proved without question that great earthquakes in a non-volcanic region occur always in the same rifts and that they occur with a certain periodicity. The Portola rift with destructive quakes of 1812, 1836, 1868 and 1906 seems to have a period of thirty to forty years."

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San Francisco Earthquake and Fire - 1906

*NOTE: Unless otherwise stated in the captions, all photographs
shown in this chapter were taken shortly after the earthquake and fire.*

Shortly after five o'clock on the morning of April 18, 1906, a series of severe earthquake shocks were experienced in an extended area of California reaching from Eureka on the north to considerably south of Monterey. Because the greatest losses were felt in San Francisco, the metropolis of this area, particularly through the terrible fire which succeeded the earthquake, the event has generally been called the "San Francisco Earthquake and Fire."

It is agreed by seismologists that the intensity of the shock at Third and Market Streets and elsewhere on similar conditions of gravel, sand or clay soil, was equivalent to an acceleration of 2 to 3.5 feet per second or approximately 1/10 g. (Reid & Wood in report of California Earthquake Commission, revised by Bailey Willis, Bulletin Seismographical Society of America, Vol. XIV, page 156). On made land such

as constituted a considerable part of the area north of Market Street and east of Montgomery, the intensities were much greater, running from 3.5 to 9.6. While this discussion is not a study of the seismology of the disaster it is important to bear these figures in mind since they indicate that a very considerable portion of the structures which survived the earthquake successfully, resisted a destructive force in some cases three times as great as the intensity employed by the insurance underwriters in their earthquake classification of May 10, 1928, for earthquake resistant design. We shall refer to this point again later.

A series of unfortunate circumstances brought it about that the earthquake was followed by a simultaneous outbreak of many fires. The water mains were broken by the shocks. The Fire Chief on whom were dependent the plans for meeting such conflagra-

No. 1—San Francisco after the earthquake and fire looking down California Street. Note the brick walls of churches standing intact and brick filler walls in the office buildings in background.

No. 2—*Fire and not earthquake caused this damage. A section of Baltimore after the great fire of 1904.*

tions, was one of the comparatively few who were killed during the earthquake itself. The shock came a few hours before breakfast, breaking chimneys and causing the breakfast fires that were shortly kindled to set fire to many buildings throughout the city. The rush of the flames was of course not immediate and it was not until from eight to eighteen hours after the shocks that many of the larger buildings were destroyed by fire. It has been estimated by various authorities that of the total loss 85 to 90% would not have occurred had not the earthquake been followed by the fire. To those who desire to study the effect of earthquakes, the fire represented another very material loss in that it obliterated many of the results of the earthquake and made it correspondingly difficult to get an accurate record of the behavior of the various classes of structures during the shake itself.

The full significance of this statement may be brought out by examining Photographs 1 and 2, the former showing a section of San Francisco after the fire looking down California Street; the latter, City of Baltimore after their disastrous conflagration of 1904. It will be seen that the Baltimore fire which was not accompanied by earthquake, levelled the city to the same condition as San Francisco. The San Francisco picture, while shocking in its devastation is interesting as it brings out the fact that every wall shown standing was of brick.

That the appearance of the city after the disaster would have been very similar if there had been no

earthquake but only the fire is brought out by comparing views of San Francisco (photo No. 1) and Baltimore after the fire of 1904 (photo No. 2). Fortunately for the record a number of photographs were taken *after the earthquake and before the fire* which clearly bring out the comparatively minor damage wrought in buildings by the shake.

The practically unharmed condition of the buildings prior to the fire is brought out in photograph No. 3. The tall central building is the new addition to the Chronicle Building; to its right is the Mutual Savings Bank. A few walls are seen to be damaged, but practically all chimneys and ornaments are in place. Certainly a small percent of total value would cover the loss shown. This district was very greatly damaged by the fire a little later.

Photograph No. 4 down Market Street during the fire, is notable for the almost complete absence of debris on the street, though all the buildings shown are brick bearing wall structures. In the left middle background appears some evidence of material in the street, otherwise all the buildings seem sound.

A picture of Union Square is shown as photograph No. 5. No sign of damage appears as the steel frames showing were buildings under construction at the time. The Spring Valley Water Company building (to which reference is later made) a bearing wall structure is in the right corner. The Claus Spreckles dome appears above the right center. The buildings shown are all brick, chiefly bearing wall structures.

No. 3—Taken between the earthquake and fire. Old and new Chronicle Buildings in center. Mutual Savings Bank at right. Note comparatively intact condition of all buildings shown including practically all chimneys. District later destroyed by fire.

No. 4—Taken during progress of fire, looking down Market Street from about Fifth. Note practical absence of debris on street, except where structures are burning.

No. 5—Union Square, taken between the earthquake and fire. No sign of earthquake damage. The two structural steel frames were buildings in course of construction, the center being the Whittell Building, and the one on extreme left being Nathan Dohrman Building.

In the congested area of the city a survey showed 2086 separate buildings with the various types of construction divided as follows:

Fire Proof	2.2%
Joisted Brick (Brick bearing walls)	68.3%
Frame	29.5%
Total	100.0%

("Insurance Engineering"—May, 1906)

This distribution of building types readily explains the impression obtained by some that the heaviest losses were in brick structures. If exactly equal proportional damage had been suffered by brick and frame structures the total damage in brick buildings would be more than twice that of frame because there were more than twice as many buildings even disregarding the unquestioned fact that the brick buildings were of at least twice the unit value of the frame buildings. When there are many evidences of damaged brick, a natural but unsound impression is given that the brick were proportionately more liable to damage than the frame structures which were so few that very little damage was noticeable.

Class "A" Structures

There were in San Francisco at the time of the disaster a considerable number of steel frame buildings with brick filler walls and with floors and partitions of incombustible material constituting what is generally known as Class A construction. While the fire showed that these were not as fire-proof as their designers intended, since their inflammable contents and wooden trim burned they were sufficiently intact to give a very clear record of their behavior during the earthquake.

The outstanding fact in this connection with these steel frame brick filler wall buildings is that not a single one of them on "firm natural ground" collapsed or was even heavily damaged by the earthquake. As this is a very strong statement, it may be important to report the statements of contemporary findings of recognized authorities.

Professor Frank Soulé, Dean of the College of Civil Engineering, University of California, states in the Engineering Record May, 1906, "The writer has visited and inspected some 25 of the principal large buildings and structures of the most modern type as

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No. 6—The tallest building in San Francisco at the time of the earthquake, the Call Building (now Spreckels Building), Third and Market Streets. Steel frame, brick filler walls and terra cotta trim. Undamaged by the earthquake. In use 1929.

No. 7—Flood Building, 870 Market Street. Steel frame, brick filler walls, granite facing, showing lack of earthquake damage although evidences of burning. In use 1929.

well as many others, and has ascertained the effects of the earthquake shock upon them previous to the fire. In every case where the structure was of Class A, steel frame with first class foundations upon good ground in natural place, no appreciable injury was done, and but for the conflagration they would now be in service. The same can be said of that class of structures resting upon excellent foundations of piles driven through made ground to a solid sub-strata.''

Edward M. Boggs, member Am. Soc. C. E., chief engineer of the Oakland Traction Consolidated and San Francisco, Oakland & San Jose Railway, states, ''I wish to give great emphasis to the fact that the magnitude of the catastrophe should be attributed principally to the conflagration which followed the earthquake. In my opinion fully 95% of the property loss in San Francisco was due to fire . . . generally speaking the buildings seriously injured were very old, of unsafe

design, improper construction or of poor materials. This earthquake was the best possible endorsement of steel frame construction for tall buildings. The skyscrapers were uninjured structurally by the shock. The damage they suffered was mainly to their finish and ornamental features. None of them collapsed; none of them contributed to the loss of life. Elevators were operated for hours or days after the shock and until the fire arrived.''

John B. Farish, the well-known Engineer, made the following interesting statement: ''Before the fire reached them, I went through a number of the large steel structures that had recently been put up. It was interesting to me to note that in no case were they seriously injured.''

But to a San Franciscan or one familiar with that beautiful city, it is hardly necessary to refer to expert opinion of this kind; merely to enumerate some of the

No. 8—Mills Building, 220 Montgomery Street. Brick walls, interior steel frame. Interior was badly damaged by fire. In use 1929.

No 9—Merchants Exchange Building, 365 California Street, showing structural condition
unaffected by earthquake. In use 1929.

buildings which in 1928 still remain among the out-
standing structures of the City and which were com-
pleted before the disaster, is convincing proof of their
stability, and the fact that their damage from earth-
quake must have been minor.

The Claus Spreckels Building (Photograph No.
6), Flood (7), Mills (8), Merchants Exchange (9),
Wells Fargo National Bank (10), Grant (11), Mu-
tual Savings Bank, Chronicle, Fairmont Hotel, St.
Francis Hotel, Pacific States Telephone (Bush St.),
(11a), and a score of other large buildings are suf-
ficient evidence.

It must be borne in mind, however, that while these
buildings varied in their exterior trim and in the de-
tails of their floors, partition walls, etc., they were all
similar in that *their filler walls were of brick or Terra
Cotta* (with the exception of some of the lower stories
where stone was used) and *none of them had rein-
forced concrete walls in their steel frames.*

An interesting comparison of the behavior of the
reinforced concrete wall and brick wall is obtainable
from a study of the Fairmont Hotel. This was a steel
frame structure with reinforced concrete in the lower
story but with brick walls, stone faced above. Photo-

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No. 10—Wells Fargo National Bank, Market and Post Streets. Interior steel frame, brick and terra cotta walls showed their resistance in spite of poor sub-soil conditions and heavy over-hanging cornices. In use 1929.

No. 11—Grant Building, 1095 Market Street, showing sound structural condition in spite of large window spaces.
Interior was destroyed by fire. In use 1929.

graph No. 12 shows the cracks that developed in the lower reinforced concrete portion while Photograph 12a shows the upper structure of steel, brick and stone practically undamaged.

In some of these buildings the brick walls were more than filler walls, being in fact full load bearing members. This was the case with the Telephone building, old Chronicle building, the Wells Fargo National Bank, and others.

From the standpoint of earthquake insurance and possible loss it is of course important that damage to the finish and interior of a building be prevented by proper earthquake construction as well as that the stability of the structure itself be assured. This subject is extremely difficult to cover from observation of the San Francisco disaster because of the fact that almost every Class A building in the City was subjected to fire. The Fairmont Hotel, however, was still uncompleted and while parts of the interior were completely destroyed by fire, the dining room floor was not burned.

No. 11a—Pacific States Telephone-Telegraph Company's main office. Brick bearing wall building un-
damaged by earthquake. As a result of their experience in San Francisco the telephone company
has continued to build regularly brick wall buildings. In use 1929.
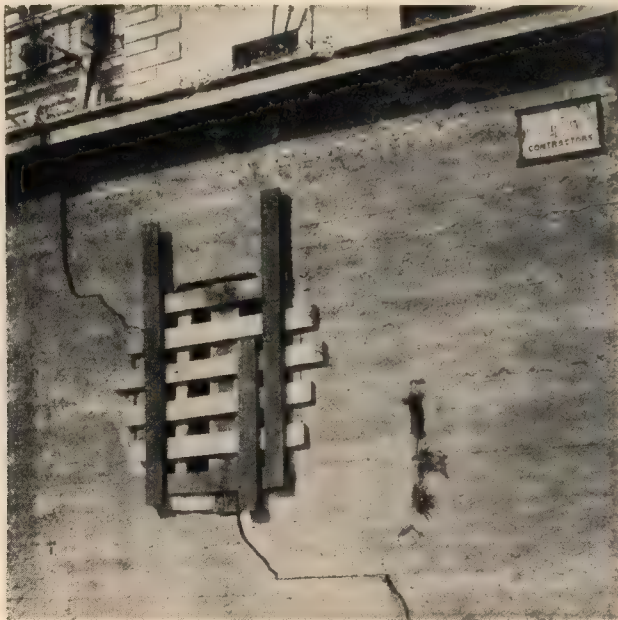
No. 12—Fairmont Hotel, showing cracks in reinforced
concrete first story walls, 26" thick.

No 12a—Portion of Fairmont Hotel, California and Powell
Streets, after fire. Steel frame, brick filler walls, stone facing,
concrete walls in lower story, metal lath partitions. Note
pile of metal lath being torn out and compare with photo-
graph No. 12, showing cracks in reinforced concrete.

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No. 14—Hollow tile partitions in earthquake and fire. A view in the Spring Valley Water Company's building (brick bearing wall). The intensity of the heat can be noted from the destruction of office furniture in the right hand corner. Tile in perfect condition from earthquake, plaster destroyed by fire.

A photograph of this room (Photograph 13), is included which shows the practically undamaged condition of the plaster and decoration. There is evident a small break in the plaster on one of the beams, which is the only apparent result. The room is shown with the work benches and scaffolds of the decorators as they were at the time of the shake.

Another piece of evidence bearing on this subject is the condition of the hollow tile partition walls in the Spring Valley Water Company Building (Photograph 14). The intensity of the heat to which these had been subjected is evident by the destruction of the plaster and the complete burning of the office furniture of which the remains are shown in the lower right hand corner. The tile partition walls, however, would seem to be entirely intact in this building and ready for use if the plaster were scraped off. As this was a building with brick walls the rigidity of the structure and the lack of damage are significant.

No. 13—A room on the dining room floor, Fairmont Hotel. This had never been occupied and the decorators were at work at the time of the disaster. Note unharmed condition of walls and decorations with the exception of small break in plaster in upper right corner. This was a steel frame, brick filler wall building.

Brick Bearing Wall Buildings

To determine the earthquake damage to non-fireproof buildings, it is important to examine the conditions in the unburned areas to ascertain the true measure of the loss.

It should be recognized at once that there were a number of cases of failures in buildings with brick bearing walls. Poor design and poor construction took their toll as always. In the case of brick buildings these faults were as usual; poor quality of mortar generally without cement, wholly inadequate anchors, ties and bonds. Such a failure for example as that at Agnew's Hospital, brought about by lack of tie between the walls and the interior wood frame, can no more be used as a general condemnation of brick work than the almost complete destruction of the observation Cyclorama in Golden Gate Park can be used to condemn concrete.

By a striking coincidence three prominent government buildings, through widely separated in the City, each came through the fire unharmed. No doubt this was due in part to their own excellent resistance but as they are among the most prominent brick buildings in the City they offer excellent evidence as to the behavior of brick structures under earthquake conditions. The first of these known as the Appraisers Warehouse is a four story structure at the corner of Sansome and Jackson Streets. This building is located on the alluvial flats of San Francisco. As brought out by Professor Bailey Willis it was subjected to considerably more than the intensity of shock than was experienced on the firm ground at Third and Market Streets (See Page 9). Nevertheless, this large structure came

through the earthquake without a crack in the walls or partitions except one or two minor cracks in arches near stairways on the upper floor. A picture of this building is shown as Photograph 15.

The second government building referred to was the United States Mint at 5th and Mission Streets. This structure also rested on soft alluvial soil, although strengthened by a pile foundation. The combination of good construction, a location at the intersection of two wide streets and an independent supply of water from an artesian well made the fire damage negligible. The construction of the building was of solid brick, faced with granite, with the floors on brick arches between steel beams. With the exception of a crack in one of the chimneys and a minor break in a wall which was weakened by a sewer under it; this building was undamaged by the earthquake. See Photograph 16. (Richard L. Humphrey—United States Geological Survey Report—pp. 42.)

The third government building was the Fontana Warehouse located outside of the fire area in the Presidio grounds. As shown by the Photograph 17, and proved by close examination of the building, this also demonstrates the qualities of good brick work.

But we by no means need rely upon the showing made by these government buildings. There were a few blocks in the vicinity of the Appraisers Warehouse referred to above and bounded roughly by Washington, Battery, Jackson and Montgomery Streets which were not burned. Fortunately photographs were taken of this section and are reproduced as (18) and (19), herewith, shown with the temporary signs used fol-

No. 15—Appraisers Stores. A four-story structure at the corner of Sansome and Jackson undamaged by
earthquake. In use 1929.

No. 16—United States Mint, Mission Street. Brick bearing walls, granite facing.
Practically undamaged. In use 1929.

No. 17—Fontana warehouse near Presidio. Refugee's camp in foreground. Undamaged by earthquake. In use 1929.

No. 18—Sansome and Jackson Streets immediately after the earthquake. All brick bearing wall buildings not burned by fire and practically all in the same condition 1929.

No. 19—Southwest corner of Jackson and Sansome looking down Jackson. These brick bearing wall buildings undamaged by earthquake, not touched by fire and still in use 1929. They were erected in the early 60's.

lowing the disaster. These show two blocks of or-dinary brick buildings of the type common through-out the older sections of the City. As indicated they were practically unharmed and serviceable; the best evidence of which is that they are almost in the same condition at the present time (January, 1929.)

Still seeking sections of the City saved from the fire we might find the cathedral at the corner of Van Ness Avenue and O'Farrel Street. This was just at the limit of the fire and indeed the tower of the cathedral be-came ignited during the conflagration, but was for-tunately extinguished. As shown in Photograph 20, this building of brick stood intact with only one small crack.

In the residential sections of the City almost un-limited numbers of buildings could be found which attest to the resistance of good brick work; such ex-amples are shown in Photographs 21 and 22, and lest it be thought that the damage to frame dwellings was only through fire, we reproduce two pictures (23) and (24), showing earthquake damage to such structures.

But if the best evidence lies in the unburned areas there is yet much to be learned from the charred ruins left by fire. The outstanding brick building of the

City and one of its most famous structures was the Palace Hotel. Gallantly resisting the flames nearly twenty-four hours, this building at last fell a victim to the fire demon and its combustible portions were destroyed. Its massive brick walls, however, stood in-tact. In spite of the fire "the wall still stands almost as good as ever." (John Stevens Sewell—United States Geological Survey, page 97.) When it was desired to build a more modern structure on the old site, continued blasting was necessary to remove the brick wall. (See Photograph 25.)

On the California Street hill stands Saint Mary's church, erected in 1854. This structure experienced the earthquakes of 1856, 1865 and 1868 as well as that of 1906. As shown in Photograph 26, the walls are intact and it is today, in 1929, faithfully serving as a house of worship.

A parallel record is that of Saint Francis church, also erected in the 50's. As shown in Photograph 27, this was constructed with very high unsupported walls having a lofty arch over the chancel. The re-markable stability of this construction, considering the natural thrust of the arch and with the large number of openings in the supporting walls, is evident from the photograph. The wooden roof and the interior of

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this church were entirely burned away, but the brick walls stood unharmed, and like the case of Saint Mary's, the church is again in regular service.

An interesting example which may be cited to show the endurance of brick work is that of the building of the California Electrical Works, shown in photograph No. 28. Surrounded by one of the most intense fires of the city, this building was saved from destruction by an independent supply of water and heavy wire glass windows. The earthquake damage

No. 20—Cathedral at Van Ness Avenue and O'Farrel Streets. Brick bearing wall building. No damage by quake, except small crack over window. Scaffolding is for repairing fire damage to tower. In use 1929.

No. 22—Brick residence unharmed by earthquake.

No. 21—Brick residence unharmed by earthquake.

No. 23—Frame buildings are not immune from earthquake damage. In addition to collapse of buildings at the left, notice buckling of buildings in right corner.

No. 24—Valencia Street showing collapse of frame buildings. Fire seen in background a few hours later wiped out all this wreckage.

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No. 25—Interior Palace Hotel showing brick walls of the first floor. Continual blasting was necessary to remove this structure when preparing for a new and enlarged building.

No. 26—Saint Mary's Church erected in 1854. Has survived 4 earthquakes and 2 fires. In use 1929.

No. 28—Brick bearing wall building of California Electrical Works preserved through fire by private water supply system. No structural damage except in walls composing roof tank pedestal.

No. 27—Interior of Saint Francis Church, built in the early 50's. Note particularly stability
of very lofty walls. In use 1929.

could therefore be carefully appraised. As stated by
the Engineering News-Record the only damage to
this 4 story brick building was in a brick cornice
around its top and some small cracks in the walls com-
posing a roof tank pedestal which carried a 50,000
gallon tank. Also a minor crack in the brick smoke-
stack.

A number of other buildings might be cited to
show the comparatively small damage caused by the
earthquake on brick buildings. The Montgomery
block on the southeast corner of Montgomery and
Washington Street, built in the late fifties and there-
fore experiencing at least 2 severe earthquakes, was
entirely unharmed and is in 1929 still in satisfactory
service.

In concluding the discussion of the behavior of
various structures in San Francisco we will present 2
more photographs of the fire in progress. No. 29 look-
ing up Market Street with the Palace Hotel in the
foreground, not only gives a spectacular view of the
fire but shows the intact condition of the Palace Hotel
and the adjoining building. The Monadnock build-
ing was in course of construction at the time and this

accounts for the canopy at the street level. We again
note the absence of debris except where the fire has
done its work. A remarkable panorama of the city in
flames taken from Mason Street is shown in photo-
graph No. 30-31. No one looking at the picture would
for a moment consider that he was viewing a city de-
stroyed by earthquake. Indeed it requires very care-
ful examination to locate the few evidences of earth-
quake damage which are to be found. A few chimneys
have collapsed but many more of them are apparently
in perfect condition. In some cases parapet walls have
been thrown down but here again most of them are
intact. Even the elaborate ornamentation of Grace
Church or Temple Emanu-el appear unharmed. It
will be remembered that with the exception of a lim-
ited number of the taller buildings, all of the masonry
structures were carried on brick bearing walls, giving
an effective answer to the question of their earthquake
resistance.

Putting together then the testimony of the experts
who have been cited and the clear photographic rec-
ord, the conclusions seem inevitably: first, that the
earthquake damage itself was comparatively small in

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No. 29—Looking up Market Street. Palace Hotel in left foreground.

the city, probably very little in excess of 5% of the total damage, as stated by Mr. Boggs above. Second, that buildings with brick bearing walls or steel frame buildings with brick filler walls not only were practically the sole structures to survive the fire, but also came through the earthquake in most cases uninjured.

This record is all the more remarkable when it is recalled that a large proportion of the brick structures were in the section of the city east of Montgomery Street on filled ground. It is elsewhere shown that the intensity of the earthquake was several times greater on soil of this character than on harder ground.

If earthquake insurance had been general in San Francisco and there had been no fire there would have been very few payments collected from companies underwriting risks on brick buildings.

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Japan Earthquake and Fire of 1923

NOTE: Unless otherwise stated in the captions, all photographs shown in this chapter were taken shortly after the earthquake and fire.

No. 32—Marunouchi Building, steel frame, brick curtain walls, terra cotta trim. One of the largest buildings in Japan, showing condition immediately after earthquake.

Japan has had the misfortune during modern times of having had the most frequent and destructive earthquakes of any civilized portion of the world. It would almost seem as though nature, jealous of the wonderful activity of man in that progressive country, decided to show that she, too, could remake things at an equal pace.

By far the most serious earthquake experienced and one which established a record for all history in extent of loss of life and also of property damage was that of September 1, 1923. As in the case of the San Francisco disaster seventeen years before, by far the greatest loss of life and property damage resulted from the fires which swept the earthquake zone following the shakes themselves. The total property damage in Japan is es-

timated at more than two billion dollars, and while the loss of life throughout the nation is variously reported, a fairly close estimate places the number of those killed at 141,720, with over 50,000 missing.

It is very evident that a factor which contributed greatly to the destruction of buildings was the unsatisfactory condition of the ground upon which most of the buildings were constructed. In the great downtown area of Tokyo, practically all foundations were on alluvial soil and many even of the larger buildings were built on piles which did not reach bedrock. While the intensity of the earthquake on firm, natural soil is estimated by seismologists as having been about 3' per second or about 1/10 g, on made ground, particularly in some portions such as Yoko-

No. 33—N. Y. K. or Yusen Building. Steel frame, brick filler walls, terra cotta trim. Condition shortly after quake. Note diagonal cracks in first floor walls. Damage to building less than 10% of value. In use 1929.

hama the intensity was many times greater, reaching as high as 16 feet or 5/10 g. Under these last conditions no structure designed by man for ordinary occupancy could be expected to survive. The most accurate reports can be had as to the City of Tokyo and this city is also the most useful for study since it contained a larger proportion of structures built on lines similar to those used in America.

A preliminary word should also be spoken relative to the types of construction used. Bricks in Japan are customarily somewhat smaller than those used in America and have compressive strengths below the standards in this country. The mortar in the more recent buildings averaged a fair quality and the workmanship on the whole probably about the same as in the United States. In the buildings constructed of steel the brick filler walls were generally of 8″ thickness, whereas in San Francisco, Los Angeles and Oakland the building codes require 12″. The Japanese building code also permitted considerably lighter bearing walls than is customary in this country and there was a notable lack of proper anchorage and ties.

A peculiar feature of construction in a number of Japanese buildings was the use of very thin flat tiles attached on the outside of the walls by a mixture of plaster of paris and cement. In a number of instances

what appears in photographs to be damage to building walls is only the breaking off of these thin wafer tiles. This is characteristic of the Japanese desire for ornamentation, many of the buildings in Japan showing an elaborateness of decoration both on the exterior and interior which would not be found in most American cities. This probably had the effect of increasing the financial loss from the earthquake.

As was probably natural in the keenly competitive building industry of today, immediately following the Japanese disaster there was a rush by the proponents of each class of building construction to prove that theirs was the only material which made a satisfactory record. Out of the mass of such claims it is difficult to arrive at the true facts. It can be asserted, however, without fear of contradiction, that where good design was carried out in good materials and workmanship, the building withstood the shock; thus the *Mitsubishi Company had constructed and owned 135 buildings in the devastated area including structures of frame, brick, steel and reinforced concrete, and not one of them was seriously damaged.* The second fact to be noted is that the records in Tokyo were, as in the case of San Francisco, badly obscured by the ensuing fire. Table I, for example, showing damage that various types of buildings suffered, as compiled

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No. 34—Tokyo-Kaijo Insurance Bldg. Steel frame, brick walls, plaster exterior. Shortly after earthquake. Damage nominal.

by the building inspection department of Tokyo, has been quoted in part to prove the heavy damage to brick buildings.

Table I
DAMAGE TO VARIOUS TYPES OF STRUCTURES
IN TOKYO
September 1, 1923
By Earthquake and Fire

	Reinforced Concrete	Steel	Brick	Masonry	Concrete
Entirely Collapsed	6 1.0%	0	24 1.7%	4 2.0%	
Half Collapsed	8 1.3%	1 1.4%	74 5.2%	5 2.5%	
Heavily Damaged	248 39.3%	25 36.7%	896 63.3%	118 59.0%	1
Slightly Damaged	221 35.0%	27 39.6%	267 18.9%	36 18.0%	1
Undamaged	148 23.5%	15 22.0%	153 10.8%	36 18.0%	1
Total	631 100%	68 100%	1414 100%	199 100%	3

Source: Building Inspection Department, Tokyo, through courtesy of H. M. Engle, Board of Underwriters, San Francisco.)

Commentators on this Table generally have omitted to mention or have given little prominence to the fact that the survey includes damage by fire. (See Hadley "How Structures withstood Japanese Earthquake and Fire.")

Many of the brick buildings were, of course, not fireproof and made no pretense at such construction any more than they do in most American cities. For this reason the Table and other similar Tables which have been much quoted do not offer a great deal of evidence regarding the strictly brick buildings, except perhaps that practically 30% of all such buildings were either undamaged or only very slightly damaged.

Dr. Imamura, the successor to the distinguished Dr. Omori at the Imperial Seismological Observatory, states (Seismological notes No. 6 Japanese Imperial Earthquake Investigation Committee.) that more than 95% of the damage in Tokyo and Yokohama was due to fire. This follows closely similar estimates, to which we have elsewhere referred, concerning San Francisco.

As a further indication of the extent of the fire losses it was reported by Robert Anderson in the Bulletin of the Seismological Society of America, page 102, Volume XV, that the Tokyo Metropolitan Police gave the total number of buildings destroyed by fire as 375,811 and by earthquake, 34,324; of which 32,375 were wooden frame buildings.

Beginning about the year 1910 there began to be a considerable amount of building in Tokyo which followed rather closely along American or European lines. During the years which followed a number of very large substantial structures were erected, some of them by American contractors. The effect of the earthquake on eighteen of the largest of these buildings was carefully studied by Frederick C. Davis, Mem. Am. Soc. C. E., who was in Japan for a number of months in 1923 and 1924. On his return in 1927-28 he made a careful check from the records of owners and architects. Mr. Davis' report is reproduced in full; our only additions having been the totals and averages.

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BUILDINGS Name of	Cost in U. S. Dollars	Damaged by Earthquake Only U. S. Dollars	Stories High	Feet High About	Size on Ground About Feet	Damage by Fire	REMARKS ABOUT STEEL FRAME		Built About Year
							Columns	Bracing	
Imperial Theatre	601,000	None	3-4	60	110x210	Yes	Normal	Normal	1910
Mitsubishi Bank, near City Hall	3,,010,000	None	4	60	185x185	No	Normal	Normal	1918
Tokyo Station	5,503,000	None	3	50	1000x150	No	Normal	Normal	1911
First Mutual Life	1,060,000	None	7	100	120x150	Yes	Normal	Normal	1912
Chiyoda Kwan	1,015,000	None	7	100	140x145	Yes	Normal	Normal	1921
Murai Bank	145,000	None	5	72	100x30-40	Yes	Normal	Normal	1901
Teikoku Seima	108,000	None	4	70	120x 30	Yes	Normal	Normal	1901
Mitsui Bank (Old)	551,000	None	3	60	150x170	Yes	Normal	Normal	1898
Mitsui Bldg. No. 3	441,000	None	8	100	75x130	Yes	Normal	Normal	1917
Mitsui Bldg. No. 2	342,000	None	6	80	110x 75	Yes	Normal	Normal	1916
Bank of Japan Annex	491,000	12,000	7	100	120x110	Yes	Normal	Normal	1912
Bank of Japan	515,000	None	3	95		Yes	Normal	Normal	1900
Mitsukoshi Store	1,562,000	None	7	100	140x350	Yes	Normal	Normal	1911
X1 Kaijo at Marunouchi	3,000,000	10,000	7	100	300x150	No	Normal	Normal	1914
X2 Tokyo Kaikan	1,500,000	140,000	5	50	140x140	No	Very Light	Almost None	1922
X3 Marunouchi	4,852,000	240,000	8	100	300x350	No	Very Light	Very Little	1922
X4 Yusen (N.Y.K.)	3,250,000	298,000	7	100	290x160	No	Normal	Very Little	1922
X5 Japan Oil (Yuraku)	2,228,000	50,000	7	100	155x145	No	Normal	Very Little	1922
Total of 6 buildings	15,321,000	750,000	av. ht. 91.6			Damage 4.9%			
Total of 12 buildings	14,753,000	None	av. ht. 85.6						

X1. KAIJO BUILDING: The $3,000,000 cost given is a conservative cost of the Building if it had been built about the year 1922. But as the Building was started in the year 1914 the actual cost was only about $750,000. The earthquake damage was practically very little, hardly perceptible, the tenants never even moved out of the Building. The Owners disbursed about $90,000—in installing new additional structural bracing and about $10,000 in repairing actual damage done by the Earthquake.

X2. TOKYO KAIKAN BUILDING: The steel frame was very light. The steel columns were so small in cross section and the bracing so little that this can hardly be classified as a steel frame building, the construction being far below both the Japanese and American standards for steel frame buildings.

About $140,000 was required for reparing actual damage done by the Earthquake, $310,000 for improvements, and about $300,000 was given the contractors to guarantee that their work would protect the Building against all damage by future earthquakes, thus making the total disbursements about $750,000 after the Earthquake.

X3. MARUNOUCHI BUILDING: The steel frame and the steel bracing were both very light especially as compared with the steel frame and bracing of new buildings of same height now under construction in Tokyo.

About $240,000 was required for repairing actual damage done by the Earthquake and about $960,000 additional was disbursed by the Owners in installing additional bracing in nearly all the frame work of the Building, thus making about $1,200,000 total disbursements after the Earthquake. The actual damage done by the Earthquake was so little that the tenants never moved out of the Building and a sight seeing visitor would see but very little or no damage either on the inside or outside of the Building.

This Building now is, and was at the time of the Earthquake and thereafter, fully occupied by tenants, no vacancies. This Building now is and always has been a very popular office building on a gigantic scale.

This report was compiled in Tokyo during the months of December, 1927, and January, February, March and April, 1928.

X4. YUSEN (N.Y.K.) BUILDING: The steel columns and girders were of normal section but the steel frame had very little bracing. About $298,000 was required for repairing the actual damage done by the Earthquake and about $402,000 was disbursed by the Owners for installing new additional bracing in the frame of the Building, thus making about $700,000 total disbursements after the Earthquake.

The actual damage done by the Earthquake was not great. With the exception of the first story, none of the tenants moved out at the time of the Earthquake and thereafter, fully occupied by tenants, no vacancies (except in the first story while undergoing repairs).

X5. YURAKU (JAPAN OIL) BUILDING: The steel columns and girders were of normal sections but had very little bracing. The damage done by the Earthquake was so little that none of the tenants moved out of the Building. The damage done to the interior plastered walls was repaired in due time.

So little damage was done to the exterior walls that the Owners have never repaired them. $50,000 would easily pay for all damage done by the Earthquake to this Building. This Building now is and always has been fully occupied by tenants, no vacancies.

Note: The particulars of all buildings mentioned in this report are based upon the following:

(a) My personal interviews with many Architects. Engineers, Building Managers and Owners during the months of December, 1927, and January, February, March and April, 1928.

(b) Within three months after the Earthquake (Sept. 1, 1923) all of the Buildings mentioned in this report were many times visited and carefully inspected by me during my stay in Tokyo at that time from December 5, 1923, to March, 1924.

My inspection and analysis was very thorough as I then expected in the near future to be called upon for a detailed report of the effect of earthquakes in general upon tall office buildings.

Report by
(Signed) FREDERICK C. DAVIS
Imperial Hotel, Tokyo, Japan
April 24, 1928.

No. 35—Japan Oil Building. Steel frame, brick filler walls faced with terra cotta. Most of damage at curved street corner. Tenants never left building during repairs.

No. 36—*View from bridge over Imperial Moat, Tokyo. N. Y. K. Building in center, Marunouchi Building extreme right, Tokyo-Kaijo Building left. All steel frame and brick filler wall structures. In center in front of N. Y. K. Building are ruins of 8-story Nagai reinforced concrete building which completely collapsed.*

The 18 buildings cited by Mr. Davis are considered the outstanding steel frame buildings of Tokyo, although there were a total of buildings which might be placed in this class of about 60. One which is particularly interesting is the Japan Industrial Bank designed by the eminent earthquake authority, Dr. Tachu Naito. Particular attention is called to this last name because of the fact that Dr. Naito is so frequently quoted as favoring only reinforced concrete walls. This building was 131 by 193 feet, of 8 stories and a basement. There were a few concrete wall bents in the building, but, as described by Dr. Naito, (Bul. Seis. Soc. of Am., Volume 17, Page 90), it was a steel frame brick filler wall structure and stood practically intact in the earthquake of 1923.

The outstanding and striking fact in Mr. Davis' report is that in the case of these 18 buildings, all of them steel with brick filler walls, the average percen-

tage of damage by earthquake was only about 2 ½ %, or in the case of the damaged buildings, excluding those undamaged, was 5%. As these buildings were in many cases erected by American contractors, they were in most respects fairly comparable with American construction except that they were built with thinner walls than would be permitted in most American cities. This would appear to be a very complete answer to the question as to the damageability of this type of construction.

Photograph 32, shows the Marunouchi Building, a steel frame and brick filler wall structure, 300x350 feet, shortly after the earthquake. The minor damage is evident.

Photograph 33, the N. Y. K. or Yusen Building, a similar structure, but with terra cotta facing.

Photograph 34, the Tokyo Kaijo Building. Was

No. 37—Mitsukoshi Department Store, steel frame, brick filler walls, terra cotta exterior. Taken considerable period after the earthquake and fire which this building survived.

similar with the exception that it had a plaster exterior over the brick wall.

Photograph 35, the Japan Oil Building located three or four blocks from the Marunouchi Building, shows the earthquake cracks, but relatively little damage which this splendid building suffered.

Photograph 36 is a view from the bridge across the moat surrounding the Imperial Palace grounds. In the center is the N. Y. K. Building. On the extreme right the Marunouchi Building, and on the left the Tokyo Kaijo Building. In the center in front of the N. Y. K. Building are shown the ruins of the Nagai Building, which was of reinforced concrete and was destroyed.

The ornate character of the construction in Japan is shown in the Mitsukoshi Department Store, Photograph 37, the Murai Bank Building, Photograph

No. 38—Murai Bank Building. Steel, brick and terra cotta. Taken somewhat after the earthquake and fire, after repairs had been made.

38, the First Mutual Insurance Building, Photograph 39, and in the Mitsubishi Bank Building, Photograph 40. These latter four pictures were taken some little time after the earthquake and do not purport to show the condition of the buildings immediately after the disaster, although, as noted by Mr. Davis, all of them were practically intact.

No. 39—First Mutual Insurance Company. Photograph taken sometime after earthquake and fire.

In some way the opinion has been circulated that the steel buildings which resisted the earthquake were only those which had reinforced concrete walls. This impression may have been erroneously drawn from the following statement of Mr. Hadley on page 12, of his report above cited. He states, "four large completed steel frame buildings in Tokyo and two practically completed, escaped without damage from the earthquake. The common characteristic of all these buildings was their complete or extensive use of reinforced concrete wall construction. These buildings are the Industrial Bank of Japan; the First Mutual Building, No. 21 Mitsubishi, and the building of Katakura & Company. These were completed and occupied. The two under construction were the Marunouchi

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No. 40—Mitsubishi Bank. Steel, brick and terra cotta.
A fine type of construction.

out the 7-story Bank of Japan Annex; the Chiyoda Kwan Building of seven stories and the Mitsukoshi Store also of seven stories, which were buildings of steel and brick filler walls which were undamaged by the earthquake.

As brought out by Mr. Davis' report, the test of damage is the cost of repair. Thus when Mr. Hadley, in the Bulletin of the Seismological Society of America, March 1924, speaks of "one steel frame building on the point of incipient collapse," the building referred to was the Tokyo Kaikan Building sometimes called the Palace Hotel Building. It is true that this building did appear to be seriously damaged, but

Hotel and the Kokko Life Insurance Building. These buildings all escaped with absolutely no damage. They were the only large steel frame buildings in Tokyo which did."

The above quotation requires qualification. As we have above indicated, the Industrial Bank of Japan was described by its own architect, Dr. Naito, as a "brick building."

Mr. Hadley continues by saying that the "Sumitomo Bank, the Mitsubishi Bank and the Railway Station were of structural steel and heavy brick or stone masonry. They were all rather low structures, however, and not to be classed with those previously mentioned." As the Sumitomo Bank was built of stone we may exclude it in this connection. The Mitsubishi Bank was of four stories 185x185 feet in plan. (Photograph 40.) While Mr. Hadley seems to disparage it as a creditable structure because it had but four stories, in spite of its having very high banking rooms, he apparently makes much of an undamaged reinforced concrete building of the same height but much less area. (Page 10, same report) and also of the 3-story Russo-Asiatic Bank Building of Yokohama. (Page 8.) In contrast to this he leaves

No. 42—Navy Department Building. Brick bearing wall construction undamaged by earthquake.

No. 41—Central Railroad Station at Tokyo. Brick bearing walls, interior steel frame. Building over 1,000 feet long.
Unharmed by earthquake or fire.

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No. 43—Metropolitan Police Board Building burning during Toyko fire. Structure was unharmed by the earth-quake and was constructed with brick bearing walls.

upon examination it was found that the damage was in two stories and most of it confined to a single story. The structural steel columns were offset several inches in places and it was necessary to cut some of the columns and shift the building back into position. Even so, this building as shown by Mr. Davis' report was repaired for under 10% of its original cost. In Tokyo as elsewhere, there were comparatively few reinforced concrete frame buildings with brick filler walls. One such example, however, was the Ogawa Tenaka Building of six stories. This building escaped without damage.

No. 45—Seiyukai Club. Marunouchi District, Tokyo. Brick bearing wall building undamaged by earthquake.

There were thus a large number of steel buildings with brick filler walls in the Tokyo earthquake, and without any reinforced concrete walls whatever, which passed through the earthquake undamaged or very slightly damaged. Construction of steel frame buildings with concrete filler walls was a comparative novelty in Japan as elsewhere and there were so few of them that there is no ground for the assertion that such buildings fared any better than the brick filler wall buildings.

No. 44—Banker's Club, Marunouchi District, Tokyo. This bearing wall building survived the earthquake undamaged.

We may again refer to Table I (Page 32) and call attention to the comparison between the steel and re-inforced concrete buildings. This comparison is on the whole fair because both classes were supposed to be fireproof so that while fire damage is not eliminated from the record it may be assumed to be equal. The first facts which are evident from this Table is that no steel building was rated as "entirely collapsed" and only one as "half collapsed." In contrast with this, six reinforced concrete buildings are shown as entirely collapsed and eight half collapsed. There can be very little doubt as to what is meant as to "entirely col-lapsed" or "half collapsed." In the classification "Heavily Damaged" and "Slightly Damaged" there would naturally be considerable variation as to per-centage of loss. The "undamaged" classification ap-

No. 46—Entrance of N. Y. K. Building, Tokyo. This shows damage to exterior terra cotta with the major ornamentation unharmed.

No. 47a—Kwannan Gateway, Asakusa, Tokyo, before disaster.

parently also includes buildings which suffered merely a minor cracking of walls.

Following the disaster a very thorough report with recommendations for future construction was prepared by W. M. Vories and Company, one of the leading engineering firms in Japan, together with engineers from Tenaka Company and the Truscon Steel Company. The report was dated September 29, 1923, and was considered by those on the ground to have been one of the most valuable of the reports rendered. After referring to the loss by fire and discussing the importance of foundations, these engineers say: "curtain walls: reinforced concrete or solid brick curtain walls are necessary up to at least the sixth floor in eight story structures. We found no buildings, even with hollow tile curtain walls, having any damage above this point." (Load bearing

No. 47c—Shows one of the small brick buildings in 47b being moved during the reconstruction program. Photo taken in 1927

clay tile was not used in Japan and the reference is to partition tile, Ed.)

"Kogyo Bank has a reinforced concrete wall up to the third floor and all corners have concrete curtain walls. All other walls are solid brick. The Ogawa and Tenaka building which shows no damage has solid brick curtain walls." After pointing out that the exterior finish of buildings cracked in proportion to the strength of the structure these engineers make this interesting comment:

"In the various buildings that were burned, the stone whether granite, marble or limestone was badly cracked and chipped by the heat. On the other hand where hard burnt tile or terra cotta was used, very little damage was done by fire."

No. 47b—The same district after disaster. Note row of small brick buildings at right of street unharmed by the earthquake or fire.

Bearing Wall Buildings

There were a large number of brick bearing wall buildings in Tokyo and their record through the earthquake is remarkable. As stated by Dr. Thomas F. Jaggar, U. S. Government Volcanologist and the representative of the Seismological Society of America, (Bulletin Vol. XIII page 144), "In Tokyo first class brick buildings four or five stories in height stood well." Some of these, like structures in San Francisco, might be classed either as bearing wall or steel frame structures. An outstanding example of this was the Central Railroad Station at Tokyo shown in Photograph 41. This is included by Davis in his list of steel frame buildings, but the walls were all bearing walls and the frame only carried the roof and interior floor loads. The building was about 1500 feet long and was absolutely unharmed by fire or earthquake. This was also true of secondary stations in Tokyo.

In general the government buildings were also brick bearing wall buildings. As an example of this type of building, Photograph 42 shows the Navy Department Building. There were a number of buildings of this type located in the district shown in Photograph 36 along the Imperial Moat. They were unharmed

No. 48—Imperial Hotel. Reinforced concrete frame, brick facing. This building was undamaged. Note also brick bearing wall building seen over the roof of the hotel occupied by a life insurance company. This was also undamaged.

by the earthquake although a number of them were burned by the fire.

A spectacular photograph which brings out this fact clearly is No. 43, showing the Metropolitan Police Building unharmed by the earthquake, but being destroyed by the conflagration.

It will be a surprise to many to know that a number of the club and newspaper buildings in Tokyo might have been located in New York or any other large American city so similar was the architecture. Photograph 44, for example, showing the Banker's Club, was a brick bearing wall building in the Marunouchi district. This is again shown in the case of the building of the Seiyukai Club, Photograph 45, whose architecture is suggestive of American colonial brick. Both were uninjured.

Bringing out the ornate character of buildings, Photograph 46 shows terra cotta at entrance of N.Y.K. Building. The small damage is evident.

Photograph 47 shows a street at the Kwannan Gate before and after the earthquake and fire. A large number of small two story brick bearing wall buildings are to be seen at the right undamaged by the earthquake and only slightly damaged by the fire.

How substantial these brick structures were, and how little damaged by the earthquake is evident from Photograph 47c from the United States Department of Commerce book on Japanese Reconstruction (page 12) published in November, 1928—and showing one of these same buildings being moved during progress of the street widening program.

One of the most spectacular buildings in Japan was the Imperial Hotel shown in Photograph 48. This was a reinforced concrete building designed by the American architect Frank Lloyd Wright. It was faced in brick throughout and was practically undamaged. In the same photograph over the roof of the hotel can be seen a life insurance building with brick bearing walls practically two stories higher than the hotel, which was also entirely undamaged.

One of the remarkable records of brick work in Japan during the earthquake was that of the railway track elevation at Tokyo. The track is carried for eight to ten miles on a series of brick arches with spans over forty feet, which all stood perfectly.

We may turn from Tokyo to Yokohama, for while the former city had structures more closely resembling American types, the effect of the earthquake in the

No. 49—Panoramic view of Yokohama after earthquake and fire. Note numbers of brick bearing wall building intact, except for the loss of combustible contents and interiors.

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No. 50——General view of Yokohama from the American Embassy. Note brick tower on City Hall in left background, also dome of Yokohama Specie Bank in extreme left corner. Both brick construction.

latter city especially on brick buildings, have been greatly misrepresented. The earthquake was more severe in Yokohama due chiefly to the very unstable soil conditions and the fire which followed the earthquake was, if anything, even more intense than at Tokyo.

Photograph 49 gives a panoramic view of Yokohama. The building in the right foreground is the Russo-Asiatic Bank, a reinforced concrete structure. With this exception practically all the walls shown are brick bearing walls. The two buildings in the right center are very similar to American buildings and the walls would seem to be absolutely intact.

Another view of Yokohama is shown in Photograph 50, which has for its central feature, the tower of the City Hall. This was a brick tower and was unharmed by either the fire or earthquake. With the exception of the building in the left foreground, all walls standing are of brick. Attention is directed to the building in the extreme left background which is the Yokohama Specie Bank. This bank is shown at closer range in Photograph 51. It was a brick bearing wall building with slate shingles on the dome. The granite which faced the bearing walls was considerably spalled by the fire but the building was entirely undamaged by the earthquake except on the

dome. Another brick bearing wall building is shown directly to the left in the same photograph.

An excellent example of earthquake resistance in Yokohama was the Y.M.C.A. building. This was of reinforced concrete frame, concrete joist floors, metal lath and plaster partitions and all brick filler walls. It had no structural damage but was badly burned in the conflagration.

Other Construction

In our review of the behavior of various buildings in Japan we have naturally paid particular attention to the behavior of the brick buildings. We have not attempted to show a complete review of the behavior of all classes of buildings as this would extend beyond the possible scope of such a study. On the other hand we have not attempted to select only the brick buildings which behaved well but have tried to pick out the most representative buildings of this type in Japan. We are perfectly aware that there were reinforced concrete buildings which stood up in the earthquake. As we have shown many of these had brick filler walls or facings. There were a great many others which failed disastrously. Photographs 52 and 53 show respectively a reinforced concrete shop building before and after the earthquake. Photograph 54 shows a reinforced concrete building of one of the elec-

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No. 51—Yokohama Specie Bank (right), Yokohama. Brick bearing wall structure. Dome constructed of
steel, covered with slate shingles.

tric companies.   Photograph 36 showed the wreck of
the Nagai Building.

The reason we insert these pictures is not in any
attempt to disparage concrete but to show that in the
case of the wreck of a concrete building, the damage
does not consist only of putting up a new building
but involves very heavy expense for clearing the site.
As we have indicated above no steel building collapsed
in the Japanese earthquake. (Or indeed in San Fran-
cisco.)   Thus it was possible to make repairs or even
to reconstruct the building at a fraction of the cost

which would have been involved if the building had
been of reinforced concrete.

When Dr. Beard was sent to Japan from the United
States to make an official study of the earthquake, he
received a detailed memorandum from Dr. R. Fujis-
awa, a seismologist. Dr. Fujisawa began his memo-
randum by stating "among all the large cities of the
world, Tokyo stands unique in this respect: It is liable
from time to time to destruction by great earthquakes
and fires which inevitably follow on their heels. In
the rebuilding of Tokyo this should receive the fore-

No. 52—Reinforced concrete shop in Tokyo, before disaster.

No. 53—Same shop after earthquake.

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No. 54—Reinforced concrete building of an electric company after earthquake.

most consideration to which all others should be subordinated.''

After thus pointing out the seriousness of the situation and indicating that the measures which he recommends are more extreme than would necessarily be applicable elsewhere, Dr. Fujisawa discusses various types of structures and other considerations for reducing the loss in case of future shakes. He has this to say regarding reinforced concrete buildings: ''The resisting power against earthquake of the reinforced concrete structure considered as a function (in the mathematical sense) of time will most probably follow a law which may be graphically represented by a curve which goes up rather steeply reaches a maximum and then gradually slopes downward. It is to be observed that the reinforced concrete buildings in Tokyo are one and all comparatively new. If in addition we think of the rusting and electrolysis of the imbedded steel rods in course of time there seems to be ample margin for doubting the accuracy of the conclusion hastily arrived at by the recent experience that .reinforced concrete structures are earthquake proof. This is the reason why I wish to limit for safety's sake, even reinforced concrete buildings to two stories.''

If we attempt to sum up the entire experience in the Japanese earthquake of 1923, we find as in other disasters no definite condemnation of any single building material or type of design. Both in the earthquake

itself and in the fire which followed frame buildings suffered by far the most serious loss, and as the greatest damage in great earthquakes has come from fire the importance of fire resistant construction was again emphasized. The value of steel frame in buildings of large size was unquestionably demonstrated. The fact that brick bearing wall buildings constructed on sound standards offer excellent resistance to both earthquake and fire was amply proved as was the low cost of repairing buildings with structural steel frame and brick filler walls. Reinforced concrete buildings, where well designed and constructed, gave a good account of themselves within limited heights. It was also made clear that in case of bad design or execution, both of which conditions were difficult to determine, in advance, the loss of life and property was terrific.

Out of the mass of figures also comes the rather startling conclusion that the direct damage due to the earthquake as distinguished from the fire was only 5% to 10% of the total valuation of the building construction in the affected areas.

The present tendency in Japanese construction is well illustrated by the recently constructed Mitsubishi Building shown in Photograph 55. This is of steel frame, concrete fireproofed granite and brick filler walls. Granite is relatively plentiful in Japan and finds favor at the present time as decorative material. There is a notable amount of unit masonry construction for the taller steel buildings.

44

No. 55—The new tendency in Japanese construction. The Mitsubishi Bulding, new structure at the right—
old structure at the left. New building of steel, granite and brick.

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No. 56—Santa Barbara immediately after the earthquake. Note comparatively small damage. Also that most of the chimneys, which were all of brick, are standing. For detailed discussion see pages 52 and 89.

Santa Barbara Earthquake - 1925

NOTE: Unless otherwise stated in the captions, all photographs shown in this chapter were taken shortly after the earthquake and fire.

About six forty-five on the morning of June 29, 1925, Santa Barbara was shaken by a severe earthquake. The seismologists classify it as rather local in character and of a severity somewhat less than that of San Francisco; on firm ground the acceleration reaching at its greatest perhaps two feet per second.

Superficial examination of the damage resulted in newspaper reports that brick buildings had shown outstanding failure while other materials were sound. As the engineers who were quickly drawn to the scene of the disaster by a desire to study the results on various types of construction made more careful examinations, it was found necessary to modify this first opinion. The engineers in general are in close agreement and we may quote some of the outstanding authorities who gave expression to their views after careful investigation.

One of the wisest summaries of the situation is by Edwin Bergstrom, the well-known Los Angeles architect, published in the Bulletin of the Allied Architects Association of Los Angeles, August 1, 1925:

"The materials of construction no doubt will be condemned because of this calamity. The failure of the spindling concrete columns and unbraced frames will be laid to the tile filler walls or to the thin brick filler walls or to the thick brick filler walls, as the case may be. It all depends on what you are attempting to prove. Brick, tile, concrete, all jumbled together in the same building; walls two feet thick tied and bonded to walls six inches thick; walls of brick 150 feet long and four stories high held together by sand and wood. Is it any wonder that tile fails, brick fails, concrete fails, steel fails, when the designer or the builder expects concrete or tile or brick to develop tensile strength or to hold together when the adhesive material is but sand that crumbles between your fingers? Materials failed not because of inherent weaknesses, but because of their unintelligent use and combination and poor workmanship in erecting them. The well-designed, honestly-built, intelligently-superintended, reinforced concrete framework did not fail in Santa Barbara or elsewhere whatever were the materials in the filler walls! The steel frame honestly riveted and tied and supported did not fail. Buildings of tile, fragile as is that material, are standing uninjured. Even the lowly concrete block can show an absolutely uncracked example in the midst of the surrounding failures in Santa Barbara. Do not let anyone persuade you that the failures were due to the materials used."

In similar vein Professor Bailey Willis and H. D. Dewell wrote a joint article in the Seismological Society of America Bulletin, page 284, Volume XV:

"Reinforced concrete buildings of superior design showed little evidence of shock, while the contrary was true where lax inspection and incompetent building were evident. Brick, stone, cement block, hollow tile and veneer walls were generally cracked or shattered. Yet in some instances brick or tile structures which had been properly braced and bonded and laid in good mortar had withstood the shock satisfactorily. Frame structures with substantial foundation showed little evidence of damage, but when foundations were inadequate or decayed the houses were wrecked by dropping to the ground."

Careful engineering study showed, to quote Paul W. Penland, former research engineer of Blue Diamond Company (largest retailer in building materials in Los Angeles, selling no brick but large quantities of cement, rock, sand and gravel) that in brick bearing wall buildings:

"(1) More than a half a hundred undamaged buildings of unit construction in the business area.
(2) A great deal of the damage, in the class of damaged buildings, was slight.
(3) The greatest amount of damage occurred on the State Street frontage of buildings, while only a portion of the rear of these buildings sustained any damage.
(4) Walls, parapet walls, cornices and pediments fell off mainly along State Street, which street runs in a north-south direction.
(5) Many walls were of 4-inch brick or brick veneer.
(6) There was a lack of sufficient anchors and ties and in many instances none at all.
(7) There was a lack of sufficient brick bond.
(8) There were buildings constructed with lime mortar, buildings constructed with Portland cement mortar, and buildings with various combinations of both."

Another summary of the situation deserves to be recalled. It is taken from a special report prepared by Messrs H. J. Brunnier, J. G. Little and T. Ronneberg, all of whom are very eminent structural engineers of San Francisco, and who have been closely identified with studies as to the results of various earthquakes. They state:

"BRICK BUILDINGS—In reference to masonry construction, the brick type is more numerous and therefore the successes and failures of brick buildings are more apparent. The failure of brick buildings in every instance is due to poor materials, poor workmanship or poor design or a combination of all three. Without exception the mortar in the destroyed buildings is of very poor quality and in many instances the particular bricks used are unsuitable for building purposes and quite generally they were laid up without header or bond courses or proper ties . . ."

No. 57—An example of standard brick building unharmed by earthquake. The Santa Barbara Y. M. C. A.

At the request of the Portland Cement Association, John C. Austin, architect, Raymond G. Osborne, testing engineer, and Paul E. Jeffers, structural engineer, each of them ranking with the best in his profession, visited Santa Barbara and rendered a report which was later published in Plastite Progress, a house organ for one of the cement companies. These experts state:

"Where brick walls were properly constructed, we found that they stood the shock practically without damage; for example, the Post Office Building and the Y. M. C. A., which withstood the shock well and suffered no damage that could not easily be repaired. . . . Generally we found the brick of good quality; and where brick construction had failed, it was due largely to the poor quality of the mortar, the workmanship, and the lack of proper anchorage. Where anchors had been used in walls that failed, they pulled out due to the lack of strength of the mortar."

It must also be borne in mind that over two-thirds of the buildings on State Street, which was the center of the shock, were of brick or brick facing. It is entirely natural that the greatest apparent damage was of this material. As is shown in the photographs which follow, brick was to a considerable extent also used as an ornamental rather than structural member as in cornices, facing, false buttresses, etc., and also in unbonded fire walls. It was in these types of construction that there was the greatest proportion of damage.

No City Inspection

Unfortunately there was no building ordinance worthy of the name in Santa Barbara prior to the earthquake and no municipal building inspection of any kind. The builder was permitted to put up anything he wished and as many of the smaller buildings were erected without any architectural assistance the structural design was frequently faulty in the extreme. Perhaps the worst fault in materials was in the mortar for which ocean sand was generally used although recognized by all as being inferior. Strength tests also show the brick in many of the buildings averaging astonishingly weak. An average of a number of samples show the older brick developing transverse strength of 600 pounds per square inch as against the Los Angeles average of 992 pounds. A compression strength of 1763 pounds per square inch as against the Los Angeles average of 2897 pounds. In these circumstances it is surprising that a very large per cent of the brick buildings were entirely undamaged.

To visualize for the reader the conditions in the unfortunate city we are presenting a number of photographs drawn from various sources. In every case they were taken within two or three days following the earthquake so that no time had been afforded for any repairs although in a few cases the debris in the streets had been cleared.

Extent of Damage

In presenting photographic evidence of the results of any earthquake or other disaster the tendency is inevitably to show the most notable damage, since showing pictures of buildings which are in every way intact and give no evidence of the event is without news value and comparatively uninteresting.

In this way the person who does not actually visit

Nos. 58 and 59—*The earthquake shows up false construction. Hollow buttresses and 4-inch brick wall with nothing to bond into in the Arlington Hotel. Note the reinforced concrete column in No. 59.*

No. 60—*The wreck of the tower, Arlington Hotel. A reinforced concrete structure designed and erected by experienced engineers.*
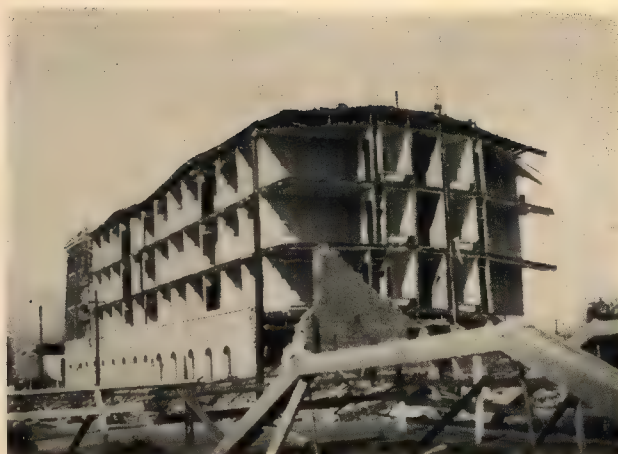
No. 61—*Unharmed brick structures on either side of a badly damaged building where a remodeled front was not tied to side walls nor anchored to roof. Note furring still in place.*

No. 62—A case of unbonded veneer walls falling when well built walls on either side were unharmed.

No. 63—Californian Hotel. Building was located on marshy ground. Walls inadequately tied and bonded were battered by movement of interior frame. This building was completely restored by the use of exterior steel frame at a total of about 40% of the original cost.

the scene of such a disaster gets a wholly distorted and exaggerated view of its extent. In the case of Santa Barbara, for example, non-technical visitors to the city on the day after the earthquake were astonished at the lack of destruction encountered with the single exception of the State Street area. A prominent eastern newspaper woman who was present is quoted as follows: "Well how does it look to you?" she was asked. "Very poor show," she remarked, "a few buildings down in the center of the town and the rest untouched." Another visitor stated, "a long ride up the wonderful mountain drive above the city showed us the Santa Barbara of our dreams, as beauti-

ful as ever; the wreckage of the main business section was quite bad enough, but what thanks we can give that the rest of the City Beautiful remained unharmed."

Mr. Oscar G. Knecht, the city building inspector for San Diego who visited Santa Barbara immediately after the earthquake confirms this view. He states:

"We hear much regarding the property loss and

No. 64—Pacific Southwest Bank, formerly Commercial Trust & Savings Building. Common brick walls, terra cotta facing. Entirely unharmed, with the exception of small crack in one rear wall.

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No. 65—Saint Francis' Hospital and Chapel after damaged filler walls and partitions had been removed. Hospital building of reinforced concrete had to be entirely demolished. It had partition tile filler walls, but the concrete frame was shattered beyond repair. The small building in the corner is a Chapel with brick bearing walls which was substantially unharmed.

the collapse and destruction of many buildings. Judging from the exaggerated reports and rumors, one would be led to believe that half of the City of Santa Barbara lies in ruins; nothing could be farther from the truth; relative to the entire city as a whole, only a small area was seriously affected, principally the retail business and commercial districts and the lower levels near the ocean. Structures on alluvial soil, sand and sedimentary fill suffered far worse than those built of solid firm clay, hardpan or rock."

Through the courtesy of City Assessor W. W. Smith of Santa Barbara and the official building records, the following tabulation is possible, covering Santa Barbara city only:

Assessment on Buildings 1925 (before
   quake) $16,727,240
      (Not including public buildings)
Value of buildings, using 40% as basis 41,720,000
Value of Public Buildings 4,280,000

Total Value of Buildings $46,000,000
Building Permits for 12 months following
   Quake .... $ 6,395,000
Increased value of buildings as shown by
   assessment, 1926 2,500,000

Loss to private buildings (difference) .... $ 3,895,000
Estimated loss to public buildings 705,000

Estimated Loss to Buildings .... $ 4,600,000
Percentage of loss to building value 10%

This does not include damage to streets, water supply, dams, public utilities, personal property, etc.

While we certainly do not mean to make light of such a disaster as that of Santa Barbara, and while we fully realize that in many individual cases the loss suffered was severe, we are desirous of bringing out that even in so heavy an earthquake the monetary

No. 66—Banca-Populare-Fugazi. Brick bearing wall, terra cotta facing. Unharmed.

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No. 67—*The Post Office seen over the ruins of the reinforced concrete San Marcos Building. The Post Office is a brick bearing wall building with interior floor loads carried by light steel frame, was unharmed except for small crack shown in left hand corner. Neither tile roof nor interior was damaged.*

damage *on the average* is not a large proportion of the total value.

An interesting confirmation of the above conclusions can be drawn from an examination of the airplane view of the damaged city presented as Photograph 56. This picture shows the section of the city which was probably at the very heart of the disturbance and surely no one in looking at it would for a moment estimate the damage as more than 10% of the total value of the buildings shown. A moment's study will indicate that the major shocks must have occurred at right angles to State Street which forms the central portion of the picture. It will be observed that the walls parallel to the street have been badly damaged in many cases, while those at right angles to the street have been for the most part unharmed.

We shall return to the discussion of some of the things shown by this picture in a later chapter, but at this point we may add that it is very evident from the photograph that the damage to chimneys was by no means as general as has been supposed. Careful computation of the chimneys shown in this photograph and two other similar photographs, showed 241 chimneys that could be counted. Of these 169 were

standing apparently intact and 72 were collapsed. In other words some slightly less than 30% of the chimneys in the center of the disturbance were destroyed. Outside of the heavily affected zone the percentage was of course much less.

Brick Structures

When we try to obtain an accurate idea of the behavior of brick structures in Santa Barbara perhaps our first step should be to distinguish between the true brick building and the building where brick because of its architectural beauty, was used to cover some less artistic material.

A splendid example of the solid brick building was the Y.M.C.A., Photograph 57. This attractive two story, tile roofed building was entirely unharmed. In contrast to this it is interesting to consider such a building as the Arlington Hotel. This structure was erroneously reported as being of brick by those who only observed the outside facing. As a matter of fact its structural members were reinforced concrete and wood with some brick. There were hollow buttresses and pillars of 4-inch brick, Photograph 58, and pilasters of concrete housed in brick shells, Photograph

No. 68—City Hall, Santa Barbara. Another example of the unharmed brick bearing wall building with tile roof. This building had interior concrete frame for floor loads only.

59. The central tower of the Arlington was of reinforced concrete, which was designed by a well-known architect and also reported to have been passed on by the Engineering Department of one of the large transcontinental railroads. It was the most seriously wrecked portion of the structure, Photograph 60.

It may be noted in passing that such a building as the Arlington would have been very properly severely penalized in any earthquake insurance schedule. Any building which mixes diverse structural materials in its supporting frame, as the Arlington mixed concrete, brick and wood, is a doubtful risk.

Another class of building in Santa Barbara which should not be confused with a brick building was the old frame building to which a brick front had been added. There were a number of such buildings on State Street from which the fronts had entirely fallen out, examples of which are shown standing between two brick buildings which were in excellent condition, Photographs 61 and 62. In the cases shown damaged building could probably be repaired at not to exceed 10% of its value. The failure of veneer facings was due to lack of any proper tie from walls to wood frame. Where the walls fell out there was often no tie at all, and at best only occasional nails to the studs. The action of such walls is no condemnation of good brick facing. In Photograph 61 the Hunt

Building and St. Charles market are old brick buildings which had recently had new face brick fronts properly applied and which stood entirely unharmed.

A building which would probably be classed as a true brick building but which did not follow strictly the ordinary design of such structures, was the California Hotel, Photograph 63. This building had an interior frame of wood which carried all the roof and floor loads. There were no adequate ties from the frame to the walls and the latter were laid in extremely poor quality of straight lime mortar. As evidence of the latter, Paul E. Jeffers, a Los Angeles engineer who had charge of the repair of this building, states that a

No. 69—State Street brick bearing wall building directly across the street from reinforced concrete Central Building, which was badly damaged.

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that it was entirely demolished, on engineering advice. The Chapel which was of ordinary brick construction had a few small cracks through the brick and mortar joints in the hexagonal tower but was substantially unharmed.

Another notable Santa Barbara bank building was the Banca Populare Fugazi, Photograph 66. This was constructed of brick with elaborate terra cotta facing and tile roofing, and except for a few brick displaced in the upper portion of a rear wall the building was entirely unharmed.

The Post Office, Photograph 67, was a brick building two stories and one-half with light interior steel frame carrying partitions and roof loads. This building was undamaged with the exception of some cracks at one corner.

Another excellent example of resistance of brick walls was in the City Hall, Photograph 68. This structure had brick bearing walls with interior concrete frame and floors. It did not have a crack.

As stated earlier in this section the undamaged or slightly damaged buildings received little public notice but are perhaps even more significant than those

Nos. 70, 71 and 72—State Street scenes shortly after earthquake. Practically all buildings shown are brick bearing wall buildings. Notice small proportion of damage.

No. 73—Brick bearing wall building on the Hotel Arlington grounds occupied by I. Magnin & Company. Building was unharmed, with the exception of small plaster crack.

contract was let for cleaning the mortar of these brick at a price of $3.00 a thousand, about a third of what such a job would cost with good mortar. In spite of the fact that this building appeared to be one of the most spectacular losses in the city it was repaired with an exterior steel frame and brick walls and put in better condition than when new for about 40% of its original cost.

The Pacific Southwest Bank (formerly Commercial Trust & Savings) Building, Photograph 64, was of common brick with terra cotta facing laid up in lime cement mortar. This was entirely unharmed with the exception of very small damage to a rear brick wall which was repaired at a cost of less than $100.00.

A very striking evidence of the value of brick construction was the Chapel of the St. Francis Hospital, Photograph 65. The hospital building proper shown in one part of the photograph was a four story reinforced concrete structure. It was so badly wrecked

No. 74—Brick oil station, showing stability of this type of construction as against stone as demonstrated by wreck of church in background.

No. 75—El Paseo Court immediately following earthquake. Bearing wall structure of load bearing hollow clay tile entirely undamaged.

that were damaged. Out of many examples that might be chosen a few such examples are shown, Photographs 69, 70, 71 and 72.

A particularly interesting case is the I. Magnin Building located right on the Arlington Hotel grounds where such serious damage occurred to an unsound building, Photograph 73. This was a two story brick building. It was undamaged, except a few plaster cracks. Illustrating the stability of the one story brick building of a type not dissimilar to the smaller brick residence is the gasoline station of the Union Oil Company, Photograph 74. That the earthquake was severe in this locality is evident from the wreck of the stone church in the background of the picture.

The First Baptist Church, Photograph 77, shows another example of beautiful and effective brick-work. There was merely a small crack on one corner of this building.

Within a block of State Street stood the Recreation Center, photograph 78, which had brick veneer walls and also the school administration building having solid brick walls, photograph No. 79, both of which were entirely undamaged.

Other Construction

It has been regretted by a number of engineers and students of earthquakes that there were no steel frame buildings in Santa Barbara; the Post Office as above stated not being an exception since the steel frame did not carry the wall loads. One of the largest buildings

in the city considered of Class A construction was designed by a very well-known Los Angeles engineer and erected under his supervision. This was the Granada Theatre Office Building, a reinforced concrete frame structure with concrete walls, the State Street frontage face brick and terra cotta exterior, Photographs 80 and 81. While it was first reported substantially unharmed, closer examination proved that concrete walls and columns in the office section of the building were badly shattered and approximately $66,000.00 has since been expended in repairs, a sum amounting to about 15% of the value of the structure. The brick walls were entirely intact.

No. 76—Poorly built frame buildings suffered as did other improper construction. A wrecked frame dwelling.

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No. 77—First Baptist Church directly across Victoria Street from the Arlington Hotel.

nation proved that concrete walls and columns in the office section of the building were badly shattered and approximately $66,000.00 has since been expended in repairs, a sum amounting to about 15% of the value of the structure. The brick walls were entirely intact.

There has been a studied attempt to show that the record of reinforced concrete buildings in Santa Barbara was a more creditable one than that of other building materials. Yet the largest individual losses were undeniably in concrete structures.

Consider the structural failures of the San Marcos Building, loss $350,000; St. Francis Hospital, loss $200,000; Central Building, loss $150,000; Granada Theater Office, loss $66,000, as well as those of Masonic Temple, Faulding Hotel, Wilson School, Lincoln School and Lathims Warehouse, besides many smaller structures. The high school, for which W. H. Weeks was the architect, was a well designed reinforced concrete structure having elaborate terra cotta ornamentation which was built under very rigid supervision, was entirely undamaged except for the displacement of a small finial.

Buildings of adequately designed and carefully constructed reinforced concrete frame with brick filler walls like the Lobero Theatre were unharmed.

The Lobero theatre, designed by George Washington Smith, architect, one block from State Street, is one of the finest examples of the earthquake resisting qualities of brick masonry bearing walls, and of brick filler walls, and load bearing hollow tile filler walls in a reinforced concrete frame used for the stage all of which was properly bonded together. There was absolutely no earthquake damage to the structure. The theatre seats about 1000 people and the stage is about 60 feet wide by 40 feet deep and about 80 feet high. A stage is admittedly one of the most difficult structures to adequately brace because the entire interior space must be entirely clear and free of any cross bracing and because of the enormous opening required over the proscenium. The method employed in constructing the stage was one insuring the maximum efficiency in the bonding between the reinforced concrete and the masonry. The brick walls of the lower section were built as panels leaving vertical chases into which the

concrete columns were poured. The spandrel beams were poured directly on top of the walls. The same method was followed in the upper portion of the stage in which load bearing hollow tile was used for the walls.

The use of load bearing hollow tile for filler walls in the upper portion of the stage is in direct accord with the practise suggested elsewhere in this report and mathematically shown to be sound as well as economical.

The photographic evidence may be concluded with a picture of the old St. Vincents Orphanage, Photograph 82. This hardy veteran was about 43 years old at the time of the disaster and had been condemned by city authorities as unsafe, nevertheless, it went through the earthquake entirely undamaged and no doubt arousing new respect by this worthy performance, was immediately reoccupied. It is a three and one-half story structure of old time brick work.

Hollow Tile In Santa Barbara

As in many other cities on the Pacific Coast, there was not adequate appreciation in Santa Barbara of the difference between load bearing hollow tile which would have passed the A.S.T.M. specifications and ordinary partition tile which is not designated or manufactured for load bearing work. Much tile was therefore used under conditions for which it was not intended and for which it should never have been employed by the builder or architect.

Yet despite this condition hollow tile in Santa Barbara had a remarkably creditable record. It may be interesting to cite some of the outstanding buildings in which hollow tile were used for bearing walls. The El Paseo Court (Photograph No. 75) was designed by the late Mr. Craig, architect, who died during its construction and was then completed under the supervision of Carleton M. Winslow, architect. This court is approximately 250 feet in one direction and about 200 feet in the other direction, being laid out in old Spanish style with large interior patio and long courtyards and is nominally two stories in height, and is a bearing wall structure built of load bearing hollow tile having a reinforcing band at the second floor and roof lines. This court was entirely undamaged.

No. 78—Santa Barbara Recreational Center Building. Entirely unharmed. Located one block from the wrecked Arlington Hotel.

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No. 79—School Administration Building. Brick bearing
wall, face brick and tile construction. With the exception
of the displacement of a few tile this building was
in perfect condition.

The Cottage Hospital with clay tile construction
and located in the zone of great disturbance, was
entirely uninjured.

There were a large number of residences in Santa
Barbara constructed of hollow tile and a large pro-
portion of these were entirely intact after the earth-
quake.

Two outstanding garages in Santa Barbara were
the Buick and Nash garages. Each of them was con-
structed with hollow tile walls and they were located
within a block of each other on the lower part of
State Street where the disturbance was very great.
They had steel roof trusses and brick pilasters.

Many other instances of satisfactory performance
of load bearing tile walls might be cited but it may
be sufficient to give the summary of a report made by
Joseph K. Moore and M. B. Reiley, who investigated
the earthquake at the request of the Hollow Building
Tile Association. They state "Hollow Building Tile
holds a unique position in the recent disaster at Santa
Barbara in that there were no failures of load bearing
walls constructed of hollow building tile."

When we turn from the buildings with tile bearing
walls to those with tile filler walls, it is not so easy
to form a correct conclusion. One of the outstanding

Nos. 80 and 81—Granada Theatre office building. Reinforced concrete frame, concrete filler walls. Photograph during
repairs showing that columns were shattered.

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No.82 —A Veteran Triumphant! The old Saint Vincent's Orphanage. Brick bearing wall building condemned before the earthquake as unsafe, but entirely undamaged by the shocks.

failures of this type of construction at Santa Barbara was the St. Francis Hospital. In this building eight inch partition tile were used in "end-construction" laid up in very soft lime mortar and expected to function as curtain walls. It is now admitted that the whole design of this beautiful building was inadequate and we have been informed that tests of the concrete showed a strength of less than 500 pounds in compression. The frame was admittedly badly designed and the whole building has subsequently been torn down. In these circumstances it seems clear that the failure of the walls was inevitable no matter what their composition. However, any calculation as to the strength requirement of the walls would have shown that the partition tile should not be used in the manner which was adopted.

In the Central Building, partition tile laid "end-construction" were used for filler walls in a reinforced concrete building. This again was a use of the material in a way not intended by the manufacturer or recommended by good authority. The frame of the building was, of course, inadequate and unless the walls had been extremely stiff and well laid, they would have been as badly damaged as was the tile. Almost identical construction was used in the Samarkand Hotel where the filler walls were made of partition tile set on end, and suffered considerable loss.

The Carrillo Hotel has offered much of interest to those who believe in the flexible lower story as discussed in a later chapter by Mr. Jeffers. In this build-ing the tile filler walls on the ground floor were very seriously damaged, but with the exception of some repairs necessitated by the action of firemen who pried open doors, there was not $500 worth of damage above the first floor. This building had tile filler walls.

We may summarize this discussion of tile filler walls by repeating that *partition tile set on end is not an adequate type of filler wall construction* and its failure under such conditions is in no sense an indictment of the material. One could as well blame concrete if an unreinforced concrete beam with a twenty foot span gave way. This construction would not have been permitted in San Francisco, Los Angeles or in Oakland.

It seems hardly necessary to state that in the buildings in which tile was used for partition purposes, and where the building itself withstood the earthquake, that the partitions were in general unharmed.

Summary

The records of brick and tile are, then, at least equally favorable with those of other materials. That there was substantial damage to brick and tile construction at Santa Barbara it would be idle to gainsay. The few outstanding failures were gross violations of elementary building principles which any inspector would have seen if there had been city inspection, in this respect presenting a different condition than concrete of which the design and construction is very difficult for the unskilled inspector to check. In Santa Barbara, the inspectors for the architects or engineers were unable, apparently, to prevent gross errors of design and construction in their material.

Certainly of buildings built of brick bearing walls (such for example as those in the Underwriters Class VII for earthquake insurance) the *average* loss was under 10%, of the building value. In the case of veneer walls which themselves probably suffered considerably over this figure, the walls were not more than 8 or 10% of the value of the building. All this in spite of the fact pointed out by all Engineers that brick work in Santa Barbara because of poor sand, mortar and inspection was notably far below the standard set by the larger cities of the state.

And in the case of hollow clay tile, where used according to the purposes for which intended, the results were most reassuring.

The Effect of Earthquake Forces on Building Construction

Spallanzani, technically trained Italian, visiting at Messina which had been destroyed by an earthquake in 1783 remarked in 1798, "that in order to meet the horizontal thrust directed at the base of a structure, it was necessary either to provide that the entire structure move as a unit under the thrust or that it have elasticity enough to carry the thrust from its own center of gravity when the shock came." This statement which is quoted with approval by Arthur L. Day, director of the Geophysical Laboratory at Washington, D. C., the outstanding institution of its kind, (Proceedings of the American Concrete Institute for 1926, page 75), concisely states the two points of view which in California have come to be known amongst engineers as the "rigidity" and "flexibility" theories.

As we review the manner in which this subject has been discussed, we are impressed with the fact that Spallanzani's statement is far closer to a correct expression than the advocates of either of the two theories have usually been willing to admit. It would be easy to cite statements of prominent engineers who have attempted to ridicule the flexibility theory by some such simple statement as that since the walls, partitions and floors of most structures are necessarily made of relatively non-flexible materials, a flexible building would inevitably suffer great loss in an earthquake, and therefore should not be considered as a practicable design. There have been on the other hand, advocates of flexibility who are certain that since no structure can be made absolutely rigid, rigidity is a sort of "Will o' the Wisp" to follow which will lead only to additional expense and greater loss in the case of any violent shock.

To us it seems that such an attitude is entirely unscientific and hardly worthy of the traditions of the engineering profession. It would be equally consistent for group of engineers to say that bridge construction should always be of the cantilever type and that arch construction was unsound; and for the engineers favoring arch construction to say that no bridge should ever be built as a cantilever. The truth is, of course, that in some conditions bridges should be designed on the cantilever system whereas in other conditions the arch type of construction is preferable.

Thus in the design of buildings to resist earthquake forces, there are undoubtedly many conditions under which the only safety lies in adequate rigidity of the entire structure. It may be, as contended by some engineers, that almost any required conditions can be met by designing the building as rigid throughout. It would seem, however, that there is an entirely tenable basis for maintaining that under certain conditions the earthquake forces may best be met by providing a certain amount of elasticity or flexibility in the lower portion of the structure. Certainly a number of structures in San Francisco and Santa Barbara could be cited in which the flexible columns on the lower floors appear to have saved the building from the heavier losses which visited their neighbors. Dr. Bailey Willis in the October, 1928, issue of the Journal of Seismological Society of America, credits the long flexible columns for saving through many earthquakes the temple of Bacchus in Palestine.

In Japan it happened that the custom prevalent in American cities of designing downtown buildings with large window space and a practical absence of exterior stiffening had not found favor at the time of the earthquake of 1923. As a result there were few, if any, buildings which would have exhibited the action of the long column first story type advocated by the proponents of the flexible theory. This has no doubt been one of the reasons through which a considerable element of Japanese structural opinion seems to favor the rigid building. No less an authority, however, than Dr. Omori, conceded by all to have been the greatest earthquake expert the world has known, was a firm believer in the possibilities of design for earthquake resistance, with flexible lower stories, and there is much talk in construction circles of Japan at present favoring this plan.

It is the endeavor of this report to present an authoritative discussion of both rigid and flexible design, hoping in this way to assist in ascertaining the practical limits of the application of each. Accordingly Mr. W. M. Butts (See Page 4) has been asked to prepare a study of design for earthquake resistance based on rigidity, and Mr. Paul E. Jeffers, prominent structural engineer, who has designed some of the most outstanding buildings in California, and who has given the study of earthquake resistant buildings many years of thought, will discuss, briefly, the advantages of flexibility.

The Design of Buildings to Resist Seismic Disturbances

By WENDELL M. BUTTS

It is helpful to recall Mallet's observation: "When one first enters one of these earthquake shaken towns he finds himself in the midst of confusion. The eye is bewildered by a city become a heap and only by gaining some commanding point * * * and observing places of greatest and least destruction and then by patient examination of many details * * * that must have produced each particular fall * * * one perceives that all confusion is superficial." And when the student of earthquake damage to buildings examines what has been written on this subject, he finds himself utterly confused by the many conflicting claims. Only by faith in the foundation of the universe; viz:—"All things occur according to fixed law," is he able to gain a proper perspective. From such a viewpoint, it becomes readily evident that many have not made "the patient examination of many details" necessary to proper conclusions; have not been sufficiently equipped with technical experience or have written to their own best interests. It was this which caused Dr. Beard, called to Japan by the Imperial Reconstruction Board shortly after the catastrophe of 1923, to say in his paper, "Analytical Report on the San Francisco Experience," that "the manufacturers of various building materials and their representatives, among the engineering profession, were eager to show that their particular kinds of materials stood the test of both earthquake and fire. They naturally cited incidents and illustrations to prove what they wanted to prove."

It is honestly desired that this Report may commend itself to consideration because of its scientific and unprejudiced slant. I was for six years structural engineer for a company first interested in the sale of reinforcing for concrete structures and then broadening its scope after the Japanese Earthquake of 1923, it included structural steel frame buildings. I am interested only in an exposition of the facts and among my present clients are those interested in all types of structures; with reinforced concrete and structural steel frames; with filler walls of concrete, brick and hollow tile; and buildings of bearing wall construction.

"Build Better Structures"

After every earthquake we have heard the statement that "good construction did not fail," and heard a plea for greater honesty and care in workmanship. Certainly these factors are important in earthquake resistance, but as may be pointed out, better construction merely along the lines of accepted practice will not necessarily suffice since the stresses set up in buildings under earthquake conditions are quite different than those taken into account in ordinary design.

Effects on Buildings in Japan—Moss

Mr. R. F. Moss, Vice-President and General Manager of the Truscon Steel Company of Japan, in his "Six Lessons of the Japanese Earthquake of 1923," gives us the best non-technical description, which has come to our attention, of the effect of an earthquake on buildings and we shall quote from it at length. Mr. Moss is an experienced graduate civil engineer, a resident of Japan for about fifteen years and the chief executive of a firm which designed several hundred structures in the devastated area. For these reasons, we believe his remarks should be given considerable weight.

"In the first place, I desire to state that my observations lead me to believe that aside from causing settlement in the case of buildings constructed on soft ground with insufficient foundations, very little damage was done by the vertical movement of the earthquake, but that the greater part of the damage was caused by the horizontal vibrations. The action of the horizontal movement in destroying or damaging buildings may be stated as follows:—A violent horizontal movement occurs in the earth and is transmitted to the foundation of the building, the extent to which the movement is transmitted to the foundation depending somewhat on the design of the foundation. The inertia of the upper portion of the building tends to keep it in its original position and the walls, columns and partitions are suddenly called upon to overcome this inertia and move the building in the direction of the quake. Successive shocks follow, each moving the foundation horizontally, forward and backward, the vertical members performing the work of moving that part of the building that is above them. As the walls, columns and partitions of each floor have only to move that part of the building above, it follows that the vertical members of the first floor will be called upon to resist forces greater than those to be resisted by the vertical members of the second floor; and that the forces to be resisted by the walls, columns and partitions of the second floor will be greater than those of the third, etc., these forces decreasing from a maximum in the first story to a minimum in the top story. It also follows that the greater the weight of the floors and roof, the greater must be the strength of the vertical members. If the combined strength of the walls and columns of each story is sufficient to resist the forces to which it is subjected, the structure will stand; if not, it will fail, in part, or collapse."

(Let me add at this point, that if any vertical members take unto themselves a greater portion of the work to be done than their strength warrants, they will be injured. They will do this if they are rigid and weak. This is critical and will be discussed later in full.)

Mr. Moss continues: "In the Marunouchi (business district of Tokyo), the greatest damage is usually found between the second and third floors and the question naturally arises as to why this is not in the first story, if that is where the greatest forces are to be resisted. The answer is, that the walls of the first story were usually strong enough to do the work that they were summoned to do, i. e. to move the building above."

Buildings in Tokyo seldom have any considerable area of glass in the first story, even on street fronts. The show window idea has not been sold to the Orient. But the walls of the second and higher stories, being made of much weaker materials, were in some cases not strong enough to move the weight above. In such cases, the maximum damage is usually observed just above the junction of the stronger and weaker walls.

"If the foregoing reasoning is correct, two important lessons follow:

"Lesson 1. Floors and roofs should be made as light as it is possible to make them, without sacrificing other equally important qualities, such as fire-proofness, permanence, etc. (and the necessary rigidity. Butts.)

"The reason for this is, of course, that if floors and roofs are light the walls and columns will have less work to do.

"Lesson 2. Walls and columns should be made strong enough to do the work which they will some day be required to do; that is, to move the weight of all the construction above.

"This should be a matter of design and not simply a question of proportioning columns to carry the vertical weight of the structure and then filling in between with brick and glass."

Methods of Design—Comparison with Dead, Live and Wind Loads

Earthquake stresses are quite different from those induced by dead, live or wind loads. Dead and live loads act vertically and ordinarily affect columns and vertical supports in compression only. Wind loads act horizontally and are the product of the vertical surface and the wind pressure and induce horizontal forces in the structure, but to an insignificant amount compared with earthquake forces. Considering a structure (150' high, 100'x100' concrete loft building) to weigh 32½ million pounds, the horizontal force induced by an earthquake of the intensity assumed by the Board of Underwriters for good ground, 0.1 g, would be 3,250,000 pounds. A horizontal wind pressure of thirty pounds per square foot would produce a force of four hundred fifty thousand pounds or less than a seventh of the force acting in an earthquake of this intensity. Also the seismic forces are set up through the centers of gravity of the structure while wind pressure is on the exterior vertical surface. For these reasons designs can not be provided to resist seismic disturbances by the simple expedient of allowing large factors of safety or heavier loadings on the usual basis of design for dead, live or wind loads. I will show that using higher live loads than are present, actually increases the danger by adding weight indiscriminately to the building, with which the earthquake may wreck the structure. The use of high wind loads is also unscientific as wind stresses and those induced by an earthquake, are not measures of one another. To produce the same horizontal loads as in the above example, it would be necessary to use a wind pressure of 217 lbs. per square foot for the particular structure, but suppose the structure was 100'x200'x13 stories, the horizontal force of the earthquake might be 6,500,000 lbs., while the wind pressure even at 217 lbs. per square foot would be but 3,250,000 lbs. Earthquake forces and wind pressures are not comparable; one depends on the weight of the structure, the other on the vertical surface area. Thus "better design and construction" on what is considered "good practice" in gravity or wind pressure, is illogical, dangerous and obviously not proper procedure.

If the wind were assumed to act on the end of a long, narrow structure it would constitute a still smaller proportion of the earthquake force in the same direction. If a heavier structure were analyzed the earthquake force would increase, but the wind force would remain the same.

Damage to Structures Not Necessarily Indications of Dishonesty

Many articles criticizing the honesty and competence of builders of structures which failed in San Francisco, in Inglewood and in Santa Barbara were grossly unfair. Where the earthquake disclosed that rubbish had been used, as was shown in the collapse of the San Francisco City Hall, to fill places which should have contained structural material, corruption is evident, but the fact that a structure collapsed did not show corruption. We are just beginning to learn to combat the conditions incident to seismic disturbances and no amount of honest materials and labor will consistently meet the issue unless the forces are first determined and provided for in the design. It is my opinion that most of the Santa Barbara and San Francisco losses were due to improper design. Some, of course, disclosed faulty materials and workmanship and practices were not always the best, though not necessarily dishonest.

General Conclusions on Types of Structures and Their Resistance to Earthquake Shocks

It has been proved beyond peradventure that a building which is practically earthquakeproof, can be constructed of almost any material. To fully understand the effect on different types of structures, it would be necessary to gather much more data on the nature of each, of the design, the materials and the

workmanship, than is possible in a report of this nature. Generalities reflect the average resistance of types of construction under the conditions of design, the character of the materials ordinarily used, the workmanship which present practices secure. But generalities must be accurately arrived at and the following are some of the factors which make many attempts to do this of doubtful value.

Design

The basis of design in the various cities of the United States varies more than is usually understood and many buildings have merely "growed" up like Topsy, and have never had the services of an engineer.

Materials

The materials which went into the 1906 buildings in San Francisco were decidedly different from those in use today. The poor sand and inferior brick in Santa Barbara, the better standards in Los Angeles and the lack of any adequate control over the quality of aggregates in San Diego, would make buildings of very different strengths in these cities, even with identical designs.

Workmanship

This varies largely with the manner in which the contracts are let and the work supervised. The character of the supervision varies greatly with the attitude of the owner, the architect and the allowance for inspection in the fee paid by the owner. It is my observation that the owner get what he pays for and if he is properly advised and pays a fair figure, his building will be well constructed.

Comparison of Actual Damage in Different Localities

Generalities which mean anything on the subject of damage to different types are difficult to prepare. There were no steel frame structures, other than shop or mill buildings in Yokohama or in Kamakura, where the Japanese earthquake of 1923 was most severe. In Yokohama most of the reinforced concrete frame structures stood, altho some were badly damaged and some well constructed brick bearing wall buildings survived undamaged. In Tokyo, in the same district, on exactly the same soil, whole blocks of brick bearing wall buildings stood, while in their midst a large reinforced concrete building, collapsed, and many structural steel frame buildings were more or less damaged. In this same district where many brick bearing wall buildings were undamaged, there were many structural steel frame buildings and reinforced concrete structures which did not show a crack. One can prove almost anything by selecting the evidence, but the fact remains that where the structure was strong enough to resist the forces set up, it survived, and where it was not strong enough, it failed in part, or collapsed if the deficiency was great. In San Francisco, there were many structural steel frame buildings with brick filler walls which were damaged little if at all, and one with walls of concrete, (not strictly a building), which was seriously damaged. On Market street stood the Palace Hotel, built of brick and next to it the Monadnock Building with a structural steel frame. The Palace Hotel was not damaged by the quake. The Monadnock was seriously injured. Such incidents were common. They do not prove anything definite, except that the builder in brick did a much better job than the designers and constructors of the structural steel frame building. And so it goes. In Santa Barbara there was not a single true steel frame building in the city, yet innumerable visitors concluded that the record of structural steel frame buildings was wonderful or a building with a structural steel frame was the answer to the earthquake problem.

The fact that no true structural steel frame building has ever collapsed in an earthquake is significant and would indicate that they have a larger factor of safety against unknown or poorly appreciated forces. Most engineers in Japan feel that buildings with reliable frames and strong walls are the type best adapted to construction to resist earthquake damage, and when properly designed and well executed will survive any shocks in which the foundation is not swept away or the problem complicated by a tidal wave.

The Period of Vibration of Structures

The effect of the vibrations on the superstructure of a building (the part above the foundation), is dependent on two factors, viz., the possibility of the synchronization of the natural period of vibration of the structure and the period of the earthquake movements and second, the strength of the component parts of the building to resist the forces to which they are subjected.

It has been definitely proven through experience with models, with seismographs and observed damage to structures that resonance between the period of the structure and the earthquake motions will build up motion through the additive effect on the oscillations to a point where the motion is so great that regardless of the strength of the building, collapse must result, if the earthquake continues for a sufficient length of time.

It has been found that buildings can be constructed to resist any movement imposed by the earthquake itself and it is advisable to erect structures whose periods of vibration are as divergent as possible from the period of the earthquake motion. The period of buildings may be determined from plans and specifications or measured directly from the completed structure and data on thousands of earthquakes indicate that the periods of destructive convulsions lie within limits sufficiently narrow to enable engineers to avoid the possibility of resonance.

In connection with this question of periods of vibration the following is quoted from a paper by Mr. Okubo, Assistant Chief Engineer of the Truscon Steel Company of Japan:——

"Assuming a perfectly rigid building, set on a rigid crust, the movement of the structure will be the same

as the crust itself. Under these conditions, the effect of the earthquake can be calculated with accuracy by the simple method of multiplying the mass of the building by the acceleration of the earthquake and designing the parts accordingly. But actually, both the crust and the building are always elastic bodies and the whole is complicated by the period of vibration of the structure" . . . "In case the period of the building is close to that of the disturbance the two may synchronize and the additive effect make the motion of the building larger and larger until the structure must collapse, if the vibrations continue."

Both of these theorems are self evident if one will realize that the motions of a block on a shaking table can only be those of the table itself, and under the second theorem, that the additive effect of small impulses to the motion of a pendulum, if of the same period as the swinging weight, is very great.

The Avoidance of Resonance of the Periods of the Building and the Quake is Fundamental

The period of vibration of a building depends on the type of construction, the character of the sub-foundation, the building materials used and the relative width to height in the direction considered, the stiffer the building the shorter the period.

The "Record of Destructive Earthquakes" shows that the period of destructive earthquakes is from 1.0 to 1.5 seconds, but it may be two seconds or more.

It is safer to construct buildings having shorter periods than the quake. This can be readily done for buildings less than a hundred and fifty or two hundred feet in height. It is considered sufficient to avoid the bracket from 1.0 to 1.5 seconds. This is not difficult to do for any height of structure.

In this connection it should be noted that a short period of natural vibration of some structures does not necessarily mean greater resistance to earthquakes than one with a longer period unless the one with the shorter period is prepared to maintain this period during a disturbance. It is quite possible for a building to have a short period but be constructed of materials which are shattered by the forces set up and the period of the structure changed due to the failure of brittle walls and inadequate bracing, Japanese experience indicates that it is desirable to use frames of great strength, not necessarily ones of great rigidity, with proper bracing due to strong walls or diagonal bracing, which will not weaken under repeated shock and allow the period to change to one which will synchronize with the trembler.

The only known method of ascertaining absolutely whether a structure has been damaged is to measure its period before and after a quake. If it has lengthened it has been ruptured, although the damage may not be visible.

Actual Design of Earthquake-Resistant Structures

From the preceding, it is evident that the design of earthquake-resistant buildings is accomplished in two steps ; viz :

First, the design of the structure for ordinary gravity loads such as is customary today, and second, the investigation of this design for its resistance to stresses induced by an earthquake of a predetermined strength.

Ordinary Gravity Design

There are certain criticisms which may be leveled against what has come to be "good practice" in building design through repetition of poorly considered methods. One of these valid criticisms is of the live loads used.

In the city of San Diego, stores must be designed with a live load of 125 pounds per square foot. In an examination of a number of stores, particularly furniture stores, which type was under consideration in a recent design, it was found that except for the loads which might result from piling rugs upon the floor, that a live load of five pounds per square foot would provide for all the merchandise and patrons. Furniture stores do not hold sales which attract large concentrations, and the particular manager consulted, indicated that not more than a dozen couples would be served at any one time in his six story 100'x100' building. Whenever an excessive live load is used in the design of a building, there is a useless expenditure of money in the construction of the slab, beams, girders, columns and footings. More than this, an unnecessary weight is added to the building, with which the earthquake may destroy the structure. *Lightness is a requisite of proper design for resistance to earthquakes.*

If a structure is designed under sane present-day building practices, with as much intelligence exercised in the details of the construction, according to standard engineering practice, with the sole exceptions noted with reference to foundations and walls of the required strength, it will survive intense earthquakes, unless it is situated immediately over a fault line or the disturbance is complicated by landslides, fissures, tidal waves or the whole vicinity drops thirty or forty feet, as it once did in Tabasco, Mexico, in 1845. *The design of seismologically safe buildings is therefore reduced to a consideration of the rigidity of vertical panels and their design for that portion of the horizontal load which falls to their lot.*

The modern frame structure is composed of a series of vertical bents of columns and girders between rigid floors. These are piled one on top of another in the form of great multi-storied buildings which ground values and convenience have created. Those who have had experience in the design and the obervation of structures during earthquakes, feel that restricting heights or the ratio of width to height, are measures of an elementary character, by no means telling the whole story. The periods of vibration of structures may be varied by introducing rigid construction so that buildings of ordinary dimensions may be constructed with a period sufficiently far from the period of destructive earthquakes to prevent resonance and structures may be erected with little reference to dimensions.

In the panels on the exterior of these multi-storied buildings, are constructed filler walls to keep out the elements, and elsewhere throughout the structure are walls for strength or merely for division purposes. If the floors are as rigid as is ordinarily the case when constructed of adequately strong material, the action of an earthquake on a building may be described as follows:

When a vibration strikes the foundations of a structure, the forces set up in a horizontal direction are a function of the weight of the building, the forces acting where the weight is located. The weight of a building above the foundation is largely distributed between the walls and columns and the floors. The exact distribution varies with each structure, depending on its use and the size and shape of the building. A warehouse carrying heavy floor loads has a greater proportion of its weight in the floors than an apartment house of the same shape and wall layout. A high building has more weight in the supporting parts than one of fewer stories and a narrow building has a lower proportion of weight in its floors than in its walls or a square building of the same floor space. The floors compose somewhere between fifty and seventy-five per cent of the weight of the building.

It is logical to combine the weight of the walls with the floors on which they rest and for convenience the weight of the story is considered as concentrated at the top of the floors, a plane readily determined by an inspection of the plans. The analysis of a structure involves a preparation of vertical cross sections and the computation of the weights of various parts of the structure. With the force of the earthquake, expressed as a percentage of gravity force, acting in each story, the forces may be computed as a part of the weight of the floors. In the preparation of this diagram it is usual to represent the force as outside of the building at the top of the floors. This is a simple method of tabulation, but it must be recognized that the force is not against the face of the structure, as with wind pressure, but in the mass of the building, wherever this is located.

TABLE 1
(All Loads in Lbs. per Sq. Foot)

Occupancy		Loads Required by 109 Cities of U. S. Average	Range	Dept. of Commerce Recommended	Investigation of Actual Loads	Uniform Loads Recommended for Seismic Design by W. M. Butts
Dwellings	First Floor	52	25-100	40. Conc. Fl. 30	Less than 40	10
	Above	50	25-100	40. Conc. Fl. 30		10
Tenements	First Floor	56	30-100	Corr. 100		10
	Above	51	30-100	40 in Rooms		10
Stores, (light)	First Floor	119	80-200	75	Furn. 10	10-50*
	Above	115	50-200	75	Groc. 25	10-50*
					Dept. 50	10-50*
Stores, (heavy)	First Floor	163	100-300	*		*
	Above	137	90-300	*		*
Warehouses	Heavy	184	100-350	*		*
	Light	138	100-300	250		*
Factories	Heavy	177	100-300	*		40
	Light	122	100-250	75		25
Roofs	Less than 20°	39	25-50	30	5	5
	Over 20°	31	20-60	20-30		5
	Movable Seats	110	70-150	100	60 heaviest	50
Assembly Halls	Fixed Seats	96	50-150	50	10 heaviest	10
Theatres	Drill	137	70-250	100	60 heaviest	50
	Dance	116	70-250	100	60 heaviest	50
Schools	Corr.	93	50-150	100		10
	Assemb.	100	60-150	100		25
	Classrooms	70	40-150	50 (Children)	10	10
				(Adults)	28	10
Office Buildings	First Floor	114	40-150	100 Corr. (2000 lbs. 2½ Sq. Ft.	7.4-14	10
	Above	70	40-150	50 in Rooms		10
Public Buildings		106	40-150	100 Corridors	Crowds 40	15
Garages	Public	126	50-250		45-60	50
	Private	74	30-200		45-60	50
Hotels	Rooms	57	30-150	40 in Rooms	4.1	5
	Corridors	88	40-150			10
Hospitals	Rooms	61	30-150	40 in Rooms	7-9	10
	Corridors	84	30-150	250 Uniform		10
Sidewalks		272	150-500	800 Concentrated		50

* Depending on use.

Wind Pressure—Less than 40 ft. high ....10 lbs. per sq. ft.
       Over 40 ft. high ....20 lbs. per sq. ft.
       Tanks, Signs ....30 lbs. per sq. ft.

The dead load must include walls, permanent partitions, framing, floors, roof, etc. Movable partitions must be provided for in any position.

Live Loads for Buildings

Table No. 1 is compiled from data from the Bureau of Standards publication, "Minimum Live Loads Allowable for Use in Design of Buildings." This Table shows the average loads required by 109 cities of the United States; the variation in these requirements, and the recommendations of the Bureau of Standards. It also shows the results of investigation by this Bureau of the *actual* loads found in the various types of structures. In the right hand column are suggested uniform live loads to be used in the investigation of buildings for resistance to earthquakes. It is believed that no higher live loads are to be expected throughout the structure at any one time. It is not contended, however, that these loads are ample for ordinary gravity design, merely that such uniform loads throughout the building are adequate for seismic investigations.

Confusion in this matter has occasioned poor reasoning as to the strength required of floors.

The increments of the horizontal force, computed as a fraction of the weight of each story, are summarized to include the increment of the floors under consideration, in order to determine the horizontal force acting in any particular plane. These horizontal forces must be carried from the roof through each story to the foundation and be absorbed by the earth. If the combined strength of the vertical parts in any story is insufficient to transmit these forces, damage will result, or if the strength of the parts is very deficient, the structure may collapse.

These forces are transmitted by any of the following methods or, by any combination, depending on the construction of the building:

1. Resistance to distortion of the walls.
2. Diagonal bracing in the planes of the vertical bents.
3. By bending and shear in the columns.

Because the walls are ordinarily more rigid than other parts, theirs will be the first attempt to transmit stresses. If they are not strong enough, other less rigid members will be called upon.

Diagonal bracing is an effective, economical and fairly well understood method for transmitting horizontal forces to the foundation. This method is extensively used in the construction of towers of all kinds, but, in buildings is seldom installed because of the difficulty experienced in avoiding the architects' layout of openings. The tendency therefore, is to use other forms such as rigid connections and knee braces in the form of portal bracing. These can usually be spaced to avoid the windows and doors.

In order to make the columns resist horizontal force, it is necessary that their connections to girders be very rigid, or the girders deeper than is usual. The ordinary standard connections are not sufficiently rigid and even with seat and top clip angles, the degree of rigidity required to utilize the strength of the columns in openings is seldom obtained. Column splices are seldom of sufficient strength to care for any considerable portion of the horizontal shear induced by

the earthquake, and detailed calculations and the observed action of buildings in great disturbances indicate clearly that most of the force must be cared for by the walls in their resistance to distortion.

Now if the floors are amply rigid, the deflection of all vertical frames in a story must be the same, irrespective of the difference of the rigidity of the frames, and the vertical panels, (frames), will take unto themselves that portion of the horizontal load corresponding to their part of the total rigidity of the vertical parts of the story. This is fundamental and the basis of all real earthquake-resistant design.

The problem then, is to calculate horizontal loads for each story, to make sure that the floors are strong enough to transmit the loads to the bents which are to resist the forces, to analyze the bents for their comparative stiffness, to equate the summation of the rigidity of the vertical panels against the horizontal load for the story, and thereby determine the amount of horizontal force to be resisted by each panel. The remaining step is to investigate the assumed panels for their unit stresses. If these are sufficient, the building will survive the earthquake without damage.

If it is then necessary to change the design of the panels because stresses in the materials are too high, it must be understood that these changes cause a redistribution of the forces.

Those engineers who design the frame work of the structure to take all the forces of the earthquake and then simply fill in the panel walls, have designed buildings in which the panel walls must be destroyed before the strong, expensive skeleton can do the work for which it was designed. This method has been characterized as the *"no collapse method,"* while the method described in this report is expected to produce a design which will suffer *no damage.*

This latter end, makes it necessary also that no part absorb more force than it can resist. In other words, that no part be too stiff for the strength which it possesses.

In order to make clear the methods which have been outlined above, the design of a structure will be undertaken and an analysis made of the stresses in its component parts. The accompanying sketch marked Fig. 1, is the floor plan of a structure 100'x100' and 150' in height, of which Fig. 2 is a vertical cross section, a typical height-limit building. (Los Angeles.) Variations from this simplified structure will be discussed after an analysis is made of the stresses induced in this building.

The design of the floors for the structure selected is as follows:

The calculations need not be shown in detail here. Suffice it to say, however, that with the live and dead loads indicated, the weight of the roof is 1,087,200 pounds, or 109 pounds per square foot. This has been taken at 110 pounds. The weight of a typical floor (dead load plus actual live load) is 1,669,000 pounds, or 167 pounds per square foot, assumed at 170 pounds.

Fig. 1

Fig. 2
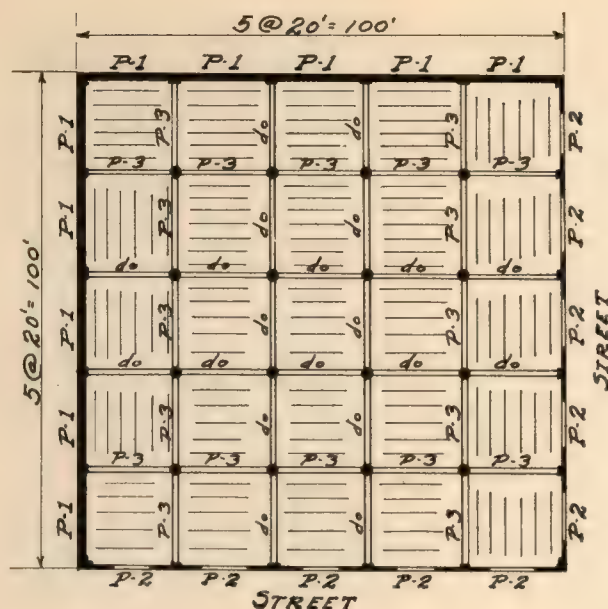
Using an intensity of one-tenth "g," which is somewhat more than that required by the Board of Fire Underwriters in the examination of buildings situated on "firm, natural ground," this gives the increments of horizontal load per floor indicated under "q" in the left hand column, on Fig. 2. The total load to be resisted by each floor is shown under "Q" in the second column from the left, on Fig. 2.

It will be noticed that the building is located on a street corner and in an earthquake in any direction, i. e., perpendicular to any face, the distortion between floors will be resisted by five vertical panels, marked "P-1," five panels, marked "P-2," and twenty interior panels between the columns, marked "P-3." These are shown in Fig 4, as solid panels, partially filled panels with openings and open panels. There are no other agencies to resist the distortion of the building in a vertical plane, and our problem is to determine the relative rigidity of these panels and equate their total rigidity against the total force to be resisted, hereby obtaining the amount to be resisted by each panel. Before we analyze the forces along these lines however, let us consider the various possible types of failure in a brick filler wall.

Effects of An Earthquake On a Vertical Panel Filled With Brick In a Structural Frame Building— Tendency of the Earthquake Forces

(1) If at right angles to the plane of the wall, the filler wall tends to fail (a) by sliding out of the panel, as a whole; (b) by shear, bricks individually slipping out of position; and (c) by bending as a floor slab.

(2) If the direction of the horizontal forces is parallel to the plane of the wall failure is by shear, by bending and by compression, due to the tendency of the quake to distort the panel.
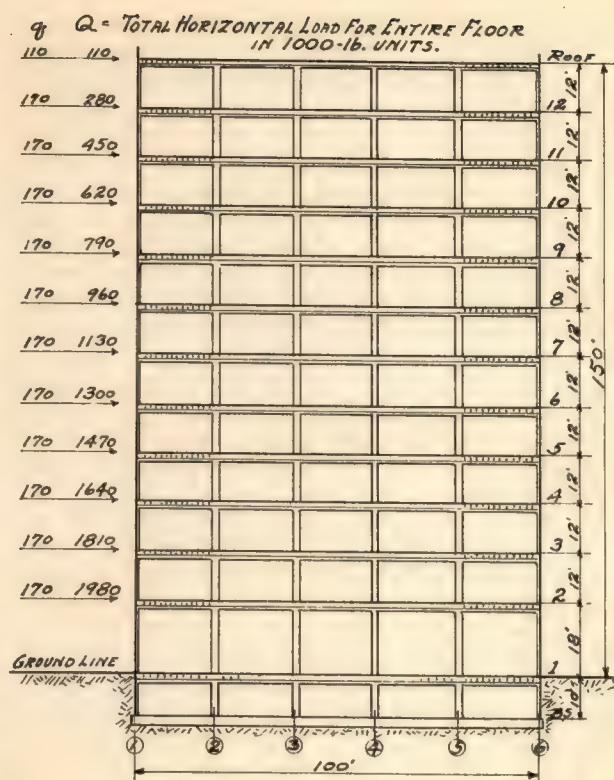
The possibilities under (1) are those of failure due to the makeup of the wall and its own weight. Under (2) the wall is assailed by extraneous forces due to the weight of the building.

(1) Parapet walls often fail from forces acting against their face. Earthquake forces set very definite and easily calculated limits to heights and thicknesses for such walls. Very few failures are due to the forces acting at right angles to filler walls.

(1a) The force necessary to throw a filler wall out of its panel intact is calculated as follows:

$$\text{Let } K = \frac{\text{Horizontal Acceleration of Quake}}{\text{Acceleration due to Gravity}} = \frac{a}{g} = 0.1$$

$$K_1 = \frac{\text{Vertical Acceleration of Quake}}{\text{Horizontal Acceleration of Quake}} = \tfrac{1}{3} K$$

Then tendency to slide due to quake =

$$\frac{K}{1-K_1} = \frac{K}{1-\tfrac{1}{3} K} = \frac{0.1}{1-(\tfrac{1}{3} \times 1/10)} = \frac{0.1}{0.966} = 0.104$$

The coefficient of friction of soft stone and steel = 0.275
The coefficient of friction of hard stone and steel = 0.20

There is therefore a factor of safety of 2. Hence while such failure is unlikely, it is well to anchor the wall adequately to the columns.

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(1b) Failure by individual shear is a special case of (1a) where the entire wall was forced out of the panel. The adhesion of the mortar to the bricks, the value of the mortar in shear and the friction of the bricks on the mortar render failure by this agency impossible, as before individual bricks could be forced out of the wall the entire panel would slip out of the frame, the coefficient of friction of brickwork on steel being less than on other brickwork.

(1c) The tendency to bend is readily reduced to figures by an examination along the same lines as the investigation of a oor slab. (Fig. 3.)
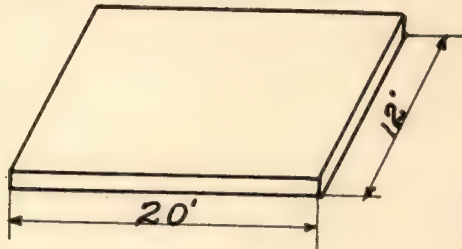
Fig. 3

Wt. per sq. ft. = 120 lbs. For K=0.1 load per sq. ft.=12 lbs.

$$\text{Two-way slab } 20^4 = 160,000$$
$$12^4 = 21,000$$
$$\overline{181,000}$$

Distribution of Load for a strip 12" wide (as inverse 4th powers)

$$\frac{21,000}{181,000} \text{ to 20' span} = 1.31 \text{ lbs.}$$

$$\frac{160,000}{181,000} \text{ to 12' span} = 10.7 \text{ lbs.}$$

$$\text{Bending Mom.} = \frac{wl^2}{8} = \frac{10.7 \times 12^2 \times 12}{8} = 2300 \text{ in. lbs.}$$

$$S = \frac{Mc}{I} = \frac{2300 \times c}{I} = \frac{2300 \times 6}{12^3} = 8 \text{ lbs., per sq. in., which is far within the actual strength.}$$

Hence there is very little possibility of a wall of this thickness failing from bending from a force at right angles to the plane of the wall.

2. We may consider next the possibility of buckling, under action of an earthquake force parallel to the wall plane.

Brick Panel Walls Will Not Fail from Buckling In An Earthquake

If the length of a compression member is more than 10 times its least lateral dimension the member is likely to bend and the intensity of the stress on the concave side will be augmented by the bending stress which arises from the eccentricity of the load. This liability to lateral bending is dependent upon the ratio of the length of the column to the least radius of gyration of the cross-section.

It is a well established principle that in short columns where the slenderness ratio is less than 125 the effect of the lateral bending is negligible, but in long columns it may be the controlling factor. Thus in a
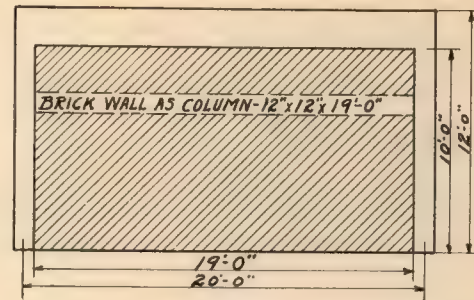
Fig. 4

panel wall 12 inches thick, 10 feet by 19 feet (Fig. 4), the slenderness ratio

$$\frac{1}{r} = \frac{10 \times 12}{.28 \times 12} = \frac{120}{3.36} = 36. \text{ which would indicate}$$

safety against bending or buckling.

Or the same conclusion may be obtained by a calculation of the actual load for bending.

By Euler's Formula for straight and homogeneous columns under axial loading:

$$\frac{P}{A} = \frac{mE}{\left(\frac{1}{r}\right)^2} \text{ (Johnson's Materials of Construction)}$$

where

P = the critical load which produces failure of column by lateral bending.
p = the critical stress in pounds per square inch.
A = area of cress section.
m = a constant depending upon end conditions.
Theoretically,
for round ends m = π^2
for fixed ends m = $4\pi^2$
Tests of actual conditions determine for square ends m = $2.5\pi^2$
Solving the above formula for P in a panel 10 ft. by 19 ft.

$$P = \frac{AmE}{\left(\frac{1}{r}\right)^2} = \frac{144 \times 2.5 \times 3.1416^2 \times 1,440,000}{\left(\frac{19 \times 12}{.28 \times 12}\right)^2}$$

P = 1,000,000 lbs., critical load on col. 12"x12".

$$p = \frac{P}{A} = \frac{1,000,000}{(12 \times 12)} = 6950 \text{ lbs. sq. in.}$$

Thus before this wall would deflect it would fail in crushing, because on the assumed column the maximum strength in direct compression would probably not exceed ¼ to ⅓ this amount.

An 8 inch wall in the same conditions gives about 3600 lbs. per sq. inch which is also well above the compression value.

Hence a brick panel wall of any customary dimensions will have a resistance to flexure several times that for direct compression on the assumed column from the direct shear on the panel and if it is to fail

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will do so in compression or shear and not by buckling.

It remains to consider the resistance to shear.

Relative Rigidities of Vertical Bents

The calculations which follow are for the purpose of determining the horizontal earthquake load which is to be resisted by each of the several types of vertical panels or bents of the building shown, in plan, in Fig. 1 and, in vertical cross-section, in Fig. 2. Figure 5 shows the details and the number of panels which will resist the tendency of the earthquake to distort the building in any direction.

As stated previously, the total load will be distributed among the vertical panels in proportion to their rigidities. The process of determining the relative rigidities is as follows:

The first computation is to determine the relative stiffness or the ratio of rigidity of Panels P-1 and P-2. On the basis of the dimensions and make-up shown for these two panels, the solid panel is eight times as rigid as the partially filled bent. In other words, to cause the same deflection eight times as much load must be applied to the solid panel as to the other and under the premise that all panels will be deflected the same amount it may be assumed that P-1 will absorb eight times as much horizontal shear as P-2.

The calculations on pages 69, 70 and 71 are to determine, this time by a slightly different method, the relative rigidity of P-2 and P-3. It is found that the partially filled panel is sixty times as stiff as the open bent.

Thus we have the relationship of one Panel P-1 being equal in resistance to distortion to eight P-2's and one P-2 being equal to 60 P-3's. From this we have the relationship of all three panels. If the open bents are taken as the unit of measure and assumed equal to P, then one P-2 equals 60 P's and one P-1 equals 480 P's.

The total rigidity equals five P-1 plus five P-2 plus twenty P-3 or expressed in terms of P's, as shown on Page 71, there are 2720 P's in all. From this is easily computed the value of one of the panels. One P-1 will resist 17.6% of the total horizontal force of the quake. One P-2 will resist 2.21% of the total horizontal force of the quake. One P-3 will resist 0.037% of the total horizontal force of the quake. These values emphasize the very large part of the total forces which the solid panels absorb, in this case 88.2% of the total tendency to distort. The partially filled panels must provide for 11.05% and the open panels for less than one per cent of the earthquake load.

The remaining step is to calculate the unit stresses set up in order to see that the assumed panels are not over stressed. This may be done for the solid panels with the full assurance that if they are safe the others will be adequate. If the load imposed by the roof load is investigated it will be found that the panels P-1 must be designed to care for 0.176 of the values for "Q" on Figure 2, page 66, 110,000 lbs. or 19,000

lbs. If this is assumed uniformly distributed across a horizontal section and the effect of impact be taken as increasing the stresses by fifty per cent the unit shear in the 12 inch thick solid walls of the twelfth story will be 10.1 lbs. per sq. in. Calculating the unit shear for the first story as a direct proportion the unit shear in the first story is 182 lbs. per sq. inch with the same wall thickness. In neither of these calculations was any value given to the shearing strength of the reinforced concrete around the columns or the structural steel members. As these values for unit shear in brick are safe it may be concluded that the design is satisfactory. These shear values are found to be safe. (See pages 76 and 80.) To allay any question, the method of determining the stresses is further reviewed in the pages that follow.

Details of the Calculation of the Relative Rigidities of Bents

The dimensions are as shown. The nomenclature used is standard but some explanation should be added:—

β is the value for "rigidity" usually represented by "K" in ordinary calculations of rigid bents, in which the reader will recognize the moment of inertia of the beam multiplied by the height of the column and the whole divided by the moment of inertia of the column multiplied by the span of the beam. The height of 144″ is used instead of the height from the floor to the center of gravity of the upper spandrel beam because there is a wall below the opening above. Also, for the reason that even at the corners there are walls in the adjoining panels, 240″ is considered as the span instead of the distance between the centers of gravity of the 72″ pilasters.

In the computation of the moments of inertia of the combination sections of brick, concrete and structural steel, cognizance of the relative deflection under load of these materials is essential. The ratio of unit stress to unit deformation is expressed by the modulus of elasticity of the material, sometimes, known as Young's modulus and designated as "E." The values assumed for E are the average values for structural steel, for 1:2:4 concrete and for brick masonry of ordinary construction. Tests shown in Johnson's "Materials of Construction" give individual brick values of 2,000,000 to 4,100,000 and cement mortar gives values of 2,500,000, but for average work the values commonly used in Japan, 1,440,000 seem conservative. The ratios of steel to brick of 21 and for concrete to brick of 2 seem therefore to be fair. These are necessary to reduce moments of inertia to a common basis.

In the computation of I′ and I the moments of inertia for the horizontal sections through the vertical panels P-2 and, P-1, respectively, no notice has been given the columns, but in both cases the dimensions are assumed as if the filler walls overlapped the columns. This is in the interest of brevity as the consid-

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CALCULATION OF RELATIVE PANEL RIGIDITY

PANELS

5 SOLID PANELS - P-1, 5 PARTIALLY FILLED PANELS P-2, 20 OPEN PANELS P-3.
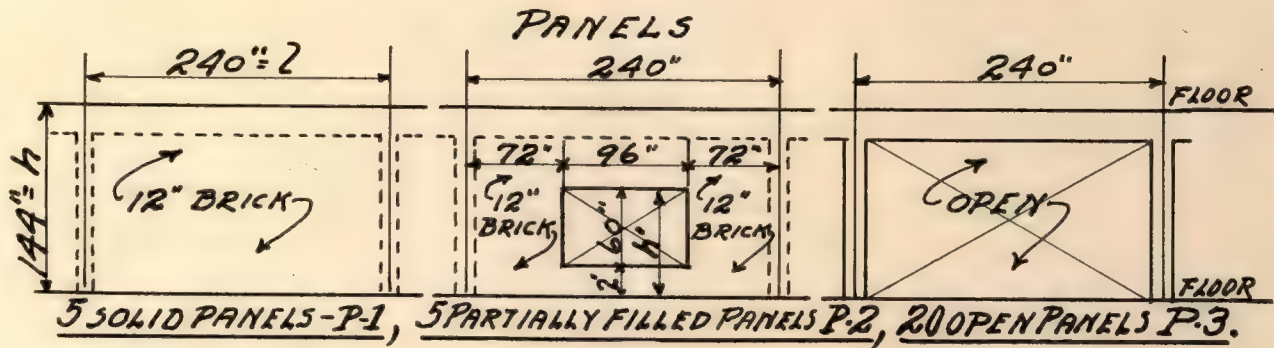
Ratio of Rigidity = $\dfrac{\dfrac{h^2}{12 I_c} \times \dfrac{2+3\beta}{1+6\beta}}{\dfrac{2.5}{A} + \dfrac{h^2}{24 I'}\left[8\dfrac{I}{I'} + \gamma^3\right]}$

P-1 to P-2 $= \dfrac{\dfrac{144^2}{12 \times 3.73 \times 10^5} 5 \times \dfrac{2+1.5}{1+3}}{\dfrac{2.5}{1728} + \dfrac{144^2}{24 \times 1.22 \times 10^7}\left[8\dfrac{1.22 \times 10^7}{1.38 \times 10^7} + \gamma^3\right]}$

$= \dfrac{\dfrac{3.5}{44.7 \times 4}}{\dfrac{8\dfrac{1.22}{1.38} + .20}{24 \times 1.22 \times 10^7}} = \dfrac{10220}{1290} = 8.$

Where, h = height as indicated = 12' = 144"

h' = " " " = 7' = 84"

I_c = Mom. of Inertia of Columns of P2 = $\dfrac{bd^3}{12} = \dfrac{12 \times 72^3}{12} = 373248.$

$\beta = \dfrac{I_b h}{I_c \, l} = \dfrac{365000 \times 144}{373248 \times 240} = 0.6$ Say 0.5 because smaller lower down.

$I_b =$
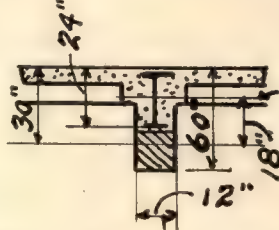
Brick Work = $36 \times 12 \times 12^2$ = 62 000
Concrete = $24 \times 12 \times 18^2 \times 2$ = 187 000
Str. Steel = $17 \times 18^2 \times 21$ = 116 000
                                          365,000

Assume E for Steel = 30,000,000 } Ratio = 21.
E " Brick = 1,440,000 } Ratio = 2.
E " Concrete = 2,880,000 }

"E" IS YOUNG'S MODULUS OF ELASTICITY.

Fig. 5

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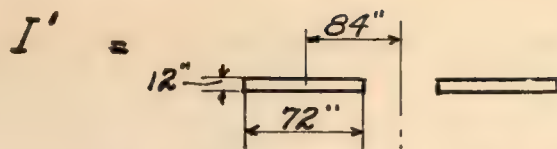
$$I' = $$

$$= 2 \times 72 \times 12 \times 84^2 = 12,200,000 = 1.22 \times 10^7$$

$$I = \frac{bd^3}{12} = \frac{12 \times 240^3}{12} = 13,800,000 = 1.38 \times 10^7$$

$$r = \left(\frac{h'}{h}\right)^3 = \left(\frac{84}{144}\right)^3 = .20$$

$$D = \text{Ratio of Rigidity} \frac{P\text{-}2}{P\text{-}3} = mn \frac{1+\beta}{1+\beta'}\left(\frac{h}{h'}\right)^3 = 85 \times .0148\left(\frac{1+41}{1+0.61}\right)\left(\frac{10}{7}\right)^3 = 96.$$

RIGID   Where $\beta' = \frac{I_b}{l} \times \frac{h}{I_c} = \frac{365000}{240} \times \frac{144}{373,248} = .606$ for P-2

SLENDER $\beta = \frac{303000}{240} \times \frac{144}{4401} = 41.$ for P-3

$$n = \frac{\beta'}{\beta} = \frac{0.606}{41} = .0148$$

$$m = \frac{I_c'}{I_c} = \frac{373248}{4401} = 85.$$

CORRECTING FOR SHEAR

$$D_c = \frac{D}{1 + 36 S_c} = \frac{96}{1 + (36 \times .018)} = \frac{96}{1.65} = 58 \text{ say } 60$$

Where, $S_c = \left(\frac{r_c}{h_c}\right)^2 = \left(\frac{.6d+4}{84}\right)^2 = \left(\frac{11.2}{84}\right)^2 = .018$

$$1 P\text{-}1 = 8 P\text{-}2$$
$$1 P\text{-}2 = 60 P\text{-}3$$
$$1 P\text{-}3 = 1P$$

$$480P - 60P - 1P$$

Fig. 6

The solid panels (P-1) and the partially filled panels (P-2) are reduced to terms of a single open panel (P-3) and the rigidity of each panel is expressed in terms of units of P, each unit of which has the strength of one P-3 panel.

TOTAL RIGIDITY =   5 P·1 + 5 P·2 + 20 P·3

$$= 2400\,P + 300\,P + 20\,P = 2720\,P$$

$1\ P\text{-}1 = \frac{1}{5}\left(\frac{2400}{2720}\right) = 17.65\%$ of HORIZONTAL FORCE

$1\ P\text{-}2 = \frac{1}{5}\left(\frac{300}{2720}\right) = 2.21\%$ "   "   "

$1\ P\text{-}3 = \frac{1}{20}\left(\frac{20}{2720}\right) = 0.0368\%$ "   "   "

PANEL· P·1
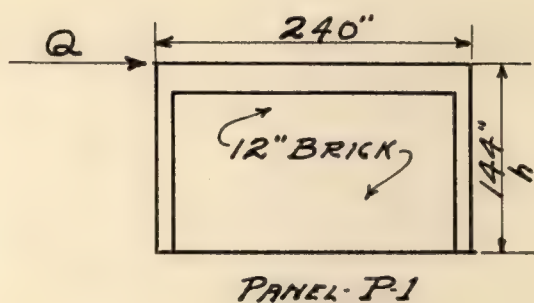
INVESTIGATION OF SOLID PANELS

AT ROOF, $Q = .1765 \times 110,000^{\#} = 19,400^{\#}$

Unit shear $= \dfrac{1.5\,Q}{A} = \dfrac{1.5 \times 19,400}{2880} = 10.1^{\#}$ per sq. in.

AT 1ST STORY $Q = .1765 \times 1,980,000^{\#} = 350,000^{\#}$

Unit shear $= \dfrac{1.5\,Q}{A} = \dfrac{1.5 \times 350,000}{2880} = 182.0^{\#}$ per sq. in.

IN THE BUILDING SHOWN THE STRESSES IN DIRECT SHEAR VARY FROM $10.1^{\#}$ PER SQ. IN. IN THE 12TH STORY TO $182.0^{\#}$ PER SQ. IN. IN THE FIRST STORY FILLER WALLS.

Fig. 7

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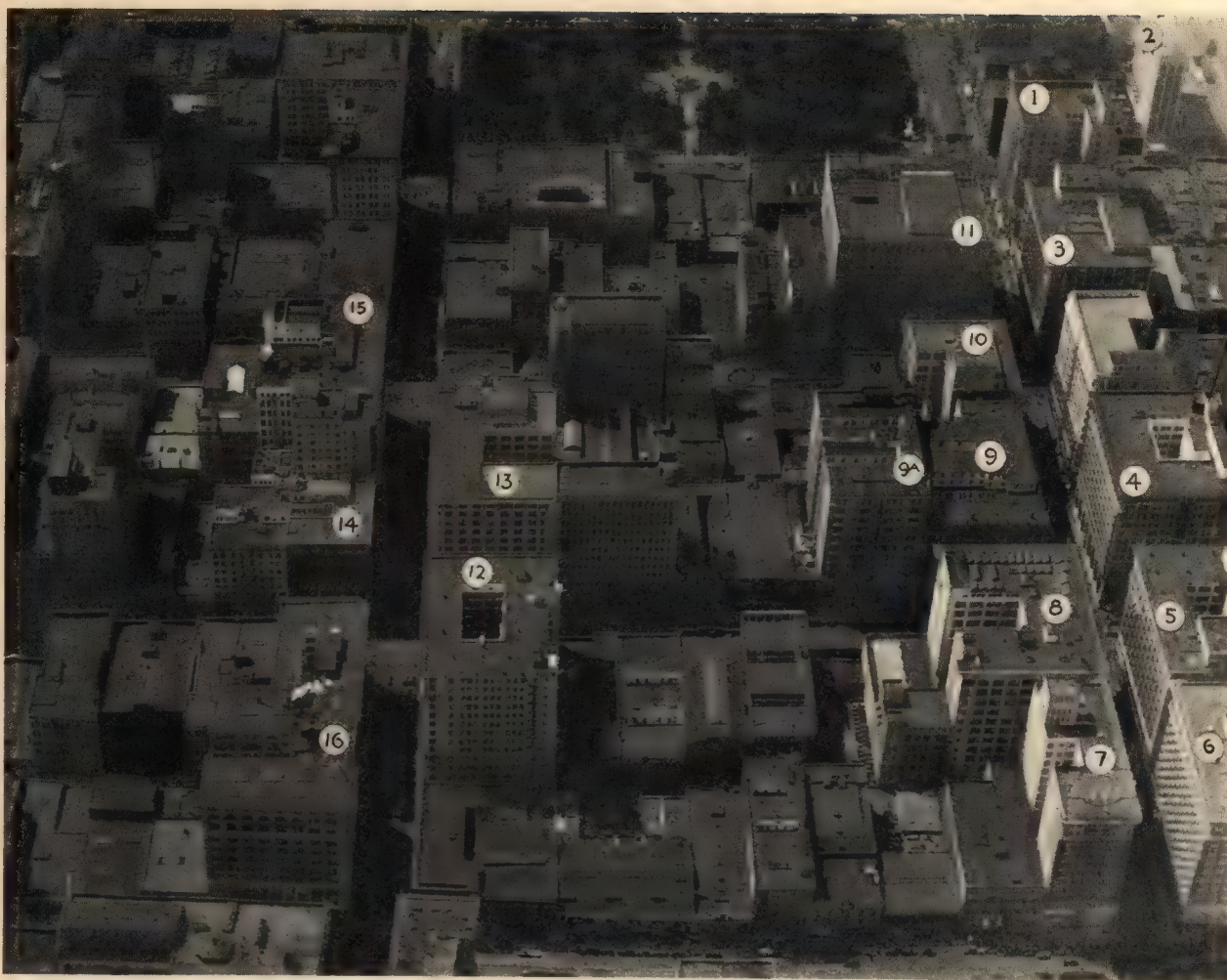
No. 83—A section of downtown Los Angeles showing typical structures. It is clear from the photograph that the building adopted for analysis in the text being 100 x 100 in plan, is more extreme than can be found in this area. Buildings of this size are regularly cut by light courts which reduces the dead load on the floor and gives additional bracing walls.

eration of the columns would make small difference in the result and the omission is on the side of conversatism.

The formula used is the general equation developed by the Japanese Imperial Earthquake Investigation Committee for determining the relative rigidity under the conditions here encountered of multiple spans and multiple stories. The second method used, that for finding the relative stiffness of panels P-2 and P-3 is a variation of the first formula.

In the above, no value has been allowed for the structural steel or the concrete fireproofing in shear, both of which elements clearly add greatly to the resistance, and take off by the amount of their shear the force on the panel walls.

The above calculations are based upon a building 100'x100' in plan dimensions and an area of 10,000 square feet; with 200 lineal feet of walls in any direction to absorb the earthquake tendency to distort. The proportion of this load assumed by the open interior

bents is so small as to be negligible. The proportion of the load which is taken by the panels on the street front is also very small. The amount to be resisted by five solid panels constitutes 88.2% of the total horizontal load.

At this juncture it must be pointed out that the conditions shown in the building analyzed, where five full panels or 100 lineal feet of solid walls resist the tendency to distort of 10,000 square feet of floor space, is an exceptional condition, and more severe a test of wall rigidity than that usually found.

Reference to Photographs 83 and 84, of Los Angeles will show no buildings with such a small proportion of wall space to floor area.

In Photograph 83, are the following structures:

1. Pershing Square Building.
2. National Bank of Commerce.
3. Metropolitan Building.
4. Citizens National Bank Building.
5. Title Insurance Building (old).

No. 84—Another view of downtown Los Angeles. Note the large number of practically solid walls giving especially effective bracing.

6. Rosslyn Hotel.
7. Rosslyn Hotel Annex.
8. Security Trust & Savings Bank Building.
9. Alexandria Hotel.
10. Title Guaranty & Trust Building.
11. Walker's Store.
12. Merchants National Bank.
13. Pacific Southwest Bank.
14. Hayward Hotel.
15. W. P. Story Building.
16. Central Building.

An examination of these will show that all have more length of wall per unit of area than the building assumed. In the structure we have analyzed there is one foot of wall length to 50 square feet of area for the open and solid panels, and 100 square feet of floor for each lineal foot of solid panel. An examination of Photographs 83 and 84, will show that there are no buildings with such a large proportion of floor area to length of wall. It may also be noted that there is a larger percentage of openings on the street fronts than in the structure which has been discussed. However, as the open panels in the assumed building provide a very small percentage of the horizontal load, this need not be an objection, provided there are other panels of sufficient strength to care for the tendency to distort.

It will be noted that some of the buildings, such as the Pershing Square Building, have no solid panels visible in the photograph, but that there are extensive wall areas with but small openings. Under these conditions, the panels with the small openings will carry somewhat less than solid panels in the assumed building, and the open panels will be required to carry a greater proportion of the load. The amount of floor area per foot of effective vertical panel is, however, much less than that in the analyzed structure. All of which would indicate that whereas, the shear in the panel walls of the building discussed reached a maximum of 182 lbs. per square inch, in the average building the shear would not reach this amount. The exact figure for any building is a matter of individual investigation, but these two photographs are submitted to show that the effective wall area is more in the average

THE NEW PACIFIC FINANCE BUILDING
LOCATED AT 621 SOUTH HOPE STREET ~ LOS ANGELES, CALIF.

CURLETT & BEELMAN
ARCHITECTS

Fig. 8

PACIFIC MUTUAL BUILDING
LOS ANGELES CALIFORNIA
W. J. DODD & WM. RICHARDS ARCHITECTS
WEST SIXTH STREET

Fig. 9
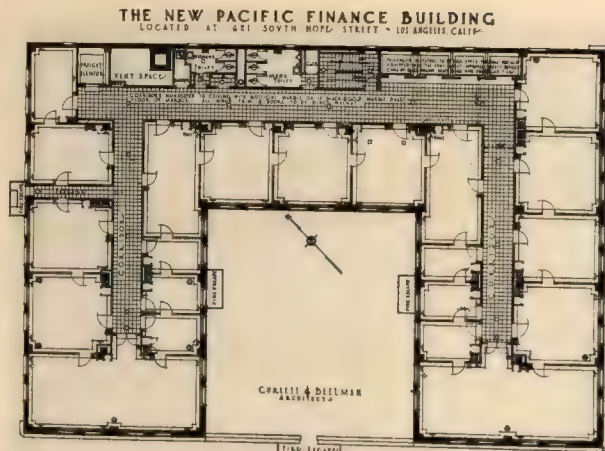
building than in the assumed structure, and, therefore the shears will usually be less.

Figures 8 and 9 are floor plans of the Pacific Finance and Pacific Mutual Buildings. These are typical modern structures. The Pacific Mutual is about ten years old and the Pacific Finance of recent construction.

An examination of the Pacific Mutual will show that we have 954 feet of exterior walls and 29,400 feet of floor space, or, approximately 62 square feet of floor space per lineal foot of wall. Thus, the walls have to resist only about 62% of the forces per foot of wall assumed in the typical structure. Notwithstanding there are no solid panels, the fact that the building is constructed with very strong corners would indicate that this building would show adequate resistance to an earthquake of the intensity assumed.

The new Pacific Finance Building has a floor area of 11,700 square feet and approximately 300 feet of strong walls in either direction. This indicates one foot of wall length to 40 square feet of area, or 40% of the amount of floor space, which must be carried by the solid walls in the assumed building. If the walls in the new Pacific Finance Building are equal in strength to the walls in the assumed building, the stresses would be only 40% of those shown for the assumed structure.

The Pacific Finance Building was very well constructed and we believe that an analysis would show that the stresses in this structure are well inside those permitted by standard practice.

It was intended that the assumed structure should show the highest reasonable stresses and for this reason a high floor area per foot of wall space was assumed. Very few structures will be found approaching the dimensions of the assumed structure, which means that ordinary buildings will have a lower stress in the walls than found in the illustration. If the material has been properly placed, we believe that structural steel frame buildings, with brick filler walls, as ordinarily designed and constructed in Pacific Coast
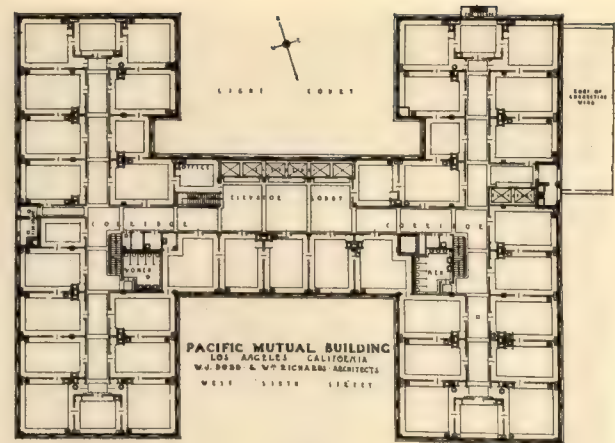
cities, will show a high resistance to earthquake damage.

Reference to airplane Photograph 85, of the heart of downtown San Francisco, which includes more tall buildings than any other section of the Coast, shows that similar conditions relative to the general plan of these buildings obtain to those in Los Angeles in that they do not have solid floor spaces in any amount proportional to 100'x100' as in the assumed structure, consequently the loads of the floors are lighter and the proportion of wall space to floor area is greater.

BRICK FILLER WALL UNDER STRESS FROM EARTHQUAKE ACTION

Fig. 10

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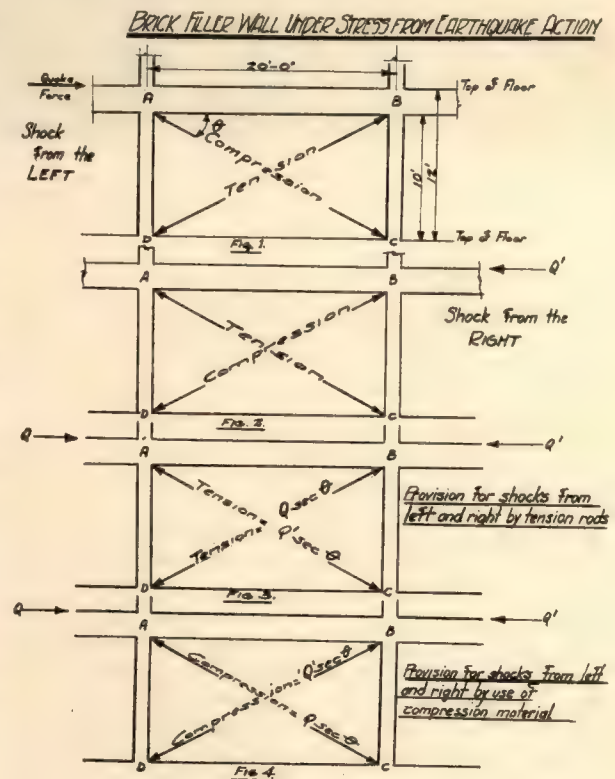
No. 85—A section of downtown San Francisco. Russ Building in foreground. In this city also the general type of building shows less extreme conditions than in the structure analyzed.

Failure of a Wall Panel

Observation of many failures of wall panels, indicates that they fail on diagonal lines between the corners. When a rectangle such as ABCD as shown in the accompanying sketches, is distorted by a force from the left at the corner (Fig. 10) "A," there is compression in the line "AC" and, when the force is applied at the corner "B" from the right, there is compression along the diagonal "BD." It follows that if in Sub. fig. 1, the compressive value of the material filling the panel is adequate, the panel will not be distorted.

Similarly, if the material in the panel is adequate to resist the compression along the line BD the panel will not be distorted from a force from the right.

Also, if tension members are provided along the line BD for the force indicated in Sub. fig. 1, and along the line AC for the force indicated in Sub. fig. 2, the panel will not be distorted. This is the usual manner of providing for stresses of this nature with tension diagonals in structural steel frames.

When the panel is filled with a compressive material, such as hollow tile, brick, concrete or reinforced concrete, and the value of this material is adequate, resistance to distortion is supplied against the forces.

If the stresses are checked under these assumptions, it will be found that for the maximum force induced in the assumed structure at points "A" or "B," the compression along the diagonals is equal to the horizontal force multiplied by the secant of the angle formed by the diagonal with the horizontal.

In the assumed structure when the maximum stresses along the diagonals of the panels are calculated into terms of compression pounds per square inch the masonry work is found to readily provide ample resistance to this compressive force.

The most important factor in determining the proper design of walls is an analysis of their methods of failure. The investigation of distortion of a rectangle indicates that for heavy loads at any corner, and the tendency to distort, compression is induced on one diagonal and tension on the other. From this,

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it may be deduced that with the changing direction of the earthquake load, both diagonals are in tension or both are in compression during the oscillations. If the rectangular panel is sufficiently strong in compression or in tension, no damage will result, as it is obviously possible to maintain the rectangular shape of the panel with a diagonal tension or compression member.

If the wall is considered as a cantilever beam, failure would be due to horizontal shear or to the opening of the brick work in tension on the loaded side. It is hardly possible that the panel can fail by bending, as the frame will prevent the lengthening of any side and distortion must take place by compression and shear, since the tendency of the rectangle when distorting to a rhomboid is to decrease its height and length. Under these conditions, the wall must fail by compression, horizontal and vertical, or by diagonal tension involving crushing, or tensile failure in a diagonal direction. To one who has observed many earthquake failures, the opening of the diagonal cracks in exterior panels of a building is an altogether too familiar sight. These failures have as often been through the brick as through the mortar joint, and we are convinced that with reasonable workmanship, 1-6 cement mortar or stronger, made more workable and stronger through the use of one part lime or by the use of diatomaceous earth and a Class "B" brick, which is the "hard well burned brick" so often specified, that filler walls of ample strength may be constructed to resist any forces to which they may be subjected in an earthquake of two or three feet acceleration on firm natural ground.

The detailed discussion of the dependable and safe values in brick and tile masonry will be found in the ensuing chapter.

Shearing Stresses

Throughout this chapter we have had frequent occasion to refer to shear. As there is considerable confusion in some quarters as to the subject, some general explanation may be helpful.

Whenever a member is subjected to loading, bending and shearing stresses are induced in the materials composing the member. The bending stresses produce tension on the opposite side from the load as at "A" and compression on the loaded side "B" (Fig. 11).
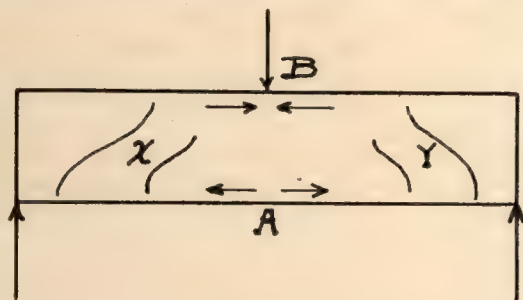
Fig. 11

Shear tends to make the particles slide upon each other. These are in two directions: in the line of action of the load and at right angles to the forces applied. In addition to the vertical and horizontal shears there are also set up diagonal stresses, known as "diagonal tension." These are of a complicated nature and hardly well understood. It has been found that reinforced concrete beams, girders and slabs have a large factor of safety against horizontal and vertical shears but the diagonal tension stresses are often critical.

While it is safe to use a working stress in direct horizontal or vertical shear of 200 pounds per sq. in. in reinforced concrete, it has been found that beams fail from diagonal tension when the direct shears are comparatively low. Because of this, it was recommended by the Joint Committee that the diagonal tension be measured by the direct shearing stresses in the member. Diagonal tension does not affect beams of homogeneous material such as steel, or wood, but is peculiar to reinforced concrete composed of materials of widely differing characteristics. It must be repeated that diagonal tension is not well understood and that the allowable stresses in direct shear are merely *measures of the more destructive stress*.

In members which do not suffer from diagonal tension working stresses for direct shear, may be used which are several times those values permitted as measures of diagonal tension.

When a rectangular filler wall is subjected to distortion, compression and tension are set up in opposite directions on the diagonals. This, however, is not the technically termed "diagonal tension," but is *direct tension* or *direct compression* in a diagonal direction and a panel may be prevented from distortion by using either tension members or compression members for the diagonals. The use of tension rods is quite universal in towers, and the Howe truss, composed of compression diagonals only, is a standard form of construction illustrating this particular point.

In addition to the compression and tensile stresses on the diagonals of the filler wall there are set up compressive stresses and shear stresses in the material.

Thus a wall may fail by crushing, tension or compression on the diagonals or by shear. If the material in the filler wall is safe against the maximum compressive load, adequate for either the compression or the tension in the diagonals or adequate for the shears, it can not fail.

The shear in a filler wall shown by calculations elsewhere in this report, is not a measure of the "diagonal tension," which must be distinguished from compressive or tensile stresses along the diagonals of the wall, and direct horizontal shear. Concrete handbooks, and general practice recommend a working stress for direct shear of more than 200 pounds. (Taylor & Thompson, Volume I, page 144.) This is with an assumed factor of safety of about 4. This value is discussed by H. M. Hadley, District Engineer, Portland Cement Association, in his pamphlet, "The Essentials of Earthquake Proof Construction," page 7, in which he states "that a concrete wall can be expected to develop an ultimate shearing unit strength of 500 to 1000 pounds per sq. inch with a 2000

pounds concrete." This seems borne out by the tests of the Mass. Institute of Technology reported in Bulletin 6, University of Illinois Engineering Experiment Station, and printed in Hool and Johnson, page 245, which average about 1200 lbs. per sq. inch.

There are three general types of shear. Vertical shear, horizontal shear, and punching shear. For the first two, true shear values of 1:2:4 concrete are about 63% of the compressive strength of the concrete and a slightly higher percentage for leaner mixes. For punching shear an allowable working stress is recommended by the Joint Committee as 120 pounds per sq. in. The direct shears, namely vertical and horizontal, are used as *measures of the diagonal tension* and as such measures in beams, girders and slabs, a stress of 40 pounds per sq. in. for members with no web reinforcement is considered adequate. For reinforced concrete beams, girders and slabs in which adequate web reinforcement is provided a stress of 60 pounds per sq. in. is allowed in the concrete. For members which do not develop diagonal tension shearing values of 200 or 300 pounds in direct shear for 1:2:4 concrete appear to have factors of safety of 4 to 8.

Reinforcing steel is used in concrete to take tensile stresses alone, and does not have any effect on the direct shearing values of the member unless used in tension as stirrups across the line of "diagonal tension." Diagonal tension cracks develop at 45 degrees, or thereabouts, to the plane of the bottom of the horizontal member and are particularly noticeable at the ends as at X and Y, in Figure 11.

NOTE: Mr. Butts' original reports to the Clay Products Institute are very much more voluminous relative to earthquake construction than could be reproduced in printed form. Several extensively treated subjects such as foundations, periods of vibration and other topics, not directly bearing on the particular problems discussed in this publication, have had to be omitted entirely, and other sections considerably abbreviated.

Some Considerations of Flexibility in Buildings

By PAUL E. JEFFERS, *Structural Engineer*

The design of buildings for communities subject to earthquake is a very large, in fact, almost unlimited subject.

To begin with, practically no two buildings are alike, therefore in order to even discuss the problem they should be grouped into types with subdivisions governed by local conditions.

It seems to me that to say that one method of structural design would apply to all these different types of divisions is rather unreasonable. A building twenty feet high and one hundred fifty feet long presents an entirely different problem than if the dimensions are reversed. A structure such as a theater stage which has very little weight for its volume and walls without openings is not subject to the same stresses that would occur in a factory building with heavy floors and loads and large window areas.

Much has been written on the so-called rigid type of design. In this type the stresses that occur are based on an assumed maximum acceleration. To carry this idea to its logical conclusion the design should be of such character that the forces are resisted by direct stress members. This is on account of the greater deformation that takes place in members if they are designed to resist these forces through bending.

If it can be designed with cross bracing in the walls or if the walls are made sufficiently strong to resist the motion and if the floors are capable of transferring their load due to inertia, to the walls, the building may rightly be considered a rigid structure.

If on the other hand it is designed to resist the action by developing bending in the members it is no longer a comparatively rigid structure but an elastic structure and no longer should the acceleration be used as a basis for figuring stresses but the deformation should be used as such basis.

In such a structure an earthquake of high acceleration but small amplitude would cause very little stress when in the rigid type the stress would be comparable to that caused by the same acceleration but with a larger amplitude.

To use a definite horizontal force in earthquake design as in design for wind stress, is a fallacy for a building that is at all flexible. This method gives a greater assumed stress the longer the columns in the first story, whereas the exact opposite is true.

In view of the fact that a majority of our taller buildings have practically no walls in the first story on street faces and cannot therefore be rigidly braced and in consideration of the fact that most of our known earthquakes have been of comparatively small amplitude, it seems to me that the logical conclusion would be to design these buildings so that we could take advantage of any elasticity that they possess to prevent damage from earthquake.

This can easily be done by making the filler walls and piers of the first story free from the columns, allowing for any deformation that would not cause excessive stress in bending in the structure. A joint

could be made at the lower side of the second story spandrels so that considerable motion could take place without visible evidence.

I am not suggesting this method as applicable to all buildings. I would advise using it only when conditions were favorable due to the physical character of the building.

There is one type of building that has no arguments in its favor and that is the one where walls and diagonal bracing are used to care for a portion of the assumed stress and the beams and columns designed to take the remainder in bending. They cannot possibly act at the same time; the bending action cannot take place until the other resistance has failed and then there is not sufficient resistance remaining to afford security for the structure.

If a study is made of buildings that have been free to move under earthquake action, it will be found that they have suffered less damage especially in the upper stories than buildings which are more rigid.

Summary and Discussion

We have had presented in the preceding papers two systems of design for earthquake resistance, the rigid and the flexible.

The former contemplates that the structure shall resist the earthquake stress by being rigid enough throughout to prevent distortion. This rigidity may be supplied by the frame, by bracing, or by masonry walls either exterior or interior, or both.

The flexible theory contemplates that the stress shall be absorbed or taken up by the deflection of the frame, the walls in the flexible portion of the building being designed to permit such deflection without damage or to crush with the thought of economical repair, the damage being confined to this zone.

It should be emphasized that the thought of Mr. Jeffers in advocating for certain types of structures, flexibility in the first story columns, is that in these conditions bending would be permitted rather than resisted and would take place in the first story columns and the connecting spandrel beams which would bend both positively and negatively. With the first story columns free to bend and the second floor spandrel beams also bending it is obvious that there is little deformation at the connections and that the standard connections designed for spandrel bearing and bending moment would in most cases be adequate to prevent rupture at these connections.

When the structural members bend they absorb a portion of the earthquake force but, of course, such bending must not be permitted to exceed the elastic limit of the steel. By absorbing the greater part of the earthquake shock in this bending in the lower story, the structure from the second floor up would be relieved of the damaging amount of the earthquake force and the frame above the second floor could be maintained as a rigid mass with thinner and lighter walls than would be the case where the first story would be designed rigidly without bending.

Mr. Jeffers undoubtedly had these thoughts in mind in designing the Carrillo Hotel in Santa Barbara, which behaved in a manner clearly demonstrating his thought, as this hotel was designed with hollow tile filler walls in the first story, which crushed when the first story took its bending. The damage was confined exclusively to the first story light walls. There was no damage above the second floor. In this connection it may be observed that the lighter weight filler walls materially decrease the dead load on the spandrel beams and the exterior columns effecting a material saving.

It is to be further noted that the two types of design indicated by Mr. Butts and Mr. Jeffers are not necessarily conflicting but are rather supplementary. Which of the methods shall be adopted depends on the character of the building being designed with the general proportions and shape of the structure and the number and character of wall openings being the governing factors.

In whatever system is selected it is fundamental that each material shall be used within its proper limits of strength. If the character of the building in respect to the possibility of adequate width to height, sufficient solid or practically solid exterior or interior bracing walls, lends itself to the rigid type of construction, this will usually be found to be the most economical. On the other hand, where it is necessary to provide a large amount of window space on all sides in the lower stories and it is not practicable to put in interior bracing walls or where the height of the building is such that to be adequate such walls would have to be of excessive thickness or extent, the flexible type of building is more appropriate.

Walls in Skeleton Frame Buildings

We have seen from the discussions by Mr. Butts and Mr. Jeffers the manner in which earthquake forces act upon buildings. In order to determine the suitability of various materials for construction in those cases where they may become subject to earthquake stress, it becomes necessary to give some consideration to the properties of the materials themselves.

Experience in brick extends over many thousands of years, during which time many of the practical problems of construction have been worked upon established lines and are generally well understood by engineers and architects. Only recently, however, it has become apparent that the more complete understanding of the engineering and technical characteristics of the material and the elements which make for strength or weakness was desirable and could be obtained in a definite way by proper technical study. During 1926 and 1927 the Bureau of Standards at Washington, with the cooperation of the Common Brick Manufacturers Association of America, conducted perhaps the most complete series of tests ever made upon building material in full size structural assemblies. No less than 168 walls of 6 by 9 feet in size and 8 to 12 inches thick were constructed, and in addition some 129 wallettes 18 by 34 inches in size.

In the construction of the walls, workmanship of both good grade, such as would be obtained by careful inspection, and poor grade, similar to that on an unsupervised contract, was employed. Individual brick of widely varying strength were separately tested, as were various combinations of mortar. The results give us a body of authoritative fact which can be safely relied upon for structural design. From the mass of data prepared by Mr. J. W. McBurney, who had charge of the tests, the following data is presented as being of the most immediate interest:

Walls 6' 0" long by 9' 0" high

No.	Kind of Brick	Mortar	Workman-ship	Strength Brick # / sq. in.	Wall Strength # / sq. in.
1	Medium (B)	Cement-lime	Poor	3370	574 minimum 586 average
2	Same	Cement	Same	Same	578 minimum 661 average
3	Same	Cement-lime	Good	3320	760 minimum 946 average
4	Hard (A)	Cement-lime	Good	8610	1840 average
5	Same	Cement	Same	Same	2710 average
		Wallettes 18" by 34" high			
6	Medium (B)	Cement	Good	3320	1344 average
7	Same construction in 6'x9' wall				1145 average
8	Same	Cement-lime	Good	3370	1033 average
9	Same construction in 6'x9' wall				946 average
10	Hard (A)	Cement	Good	8610	3530 average
11	Same construction in 6'x9' wall				2710 average

With poor workmanship a medium grade of brick testing 3370 pounds (1) in compression (falling in grade B in the A. S. T. M. specifications), laid up with cement-lime mortar, a minimum compressive strength of 574 pounds per square inch and an average compressive strength of 586 pounds per square inch was developed in the wall. With the same workmanship and a one to three cement mortar (2) a minimum of 578 pounds and an average of 661 pounds was attained. The numbers in parenthesis refer to the examples in the previous table.

With good workmanship a stronger grade of brick testing 8610 pounds (4) in compression and thus falling in grade A of the A. S. T. M. specifications, laid with cement-lime mortar, an average wall strength of 1840 pounds per square inch was obtained, while with the same brick and a one to three cement mortar (5) an average of 2710 pounds per square inch in wall strength was developed.

While these tests are primarily concerned with strength in compression, taken together with other studies they throw a very valuable light upon the tensile and shearing strength of brick masonry. McBurney notes that except where very little or very poor mortar was used, typical failure of brick masonry was by the crushing of the brick. He also noted that a mortar proportioned of 1 : 1 : 6 or stronger was apparently uninjured. With these facts thoroughly established with a suitable mortar and with full mortar beds and joints, the failure will be through the brick and the brick value may be used in determining the strength of the masonry.

McBurney also brings out clearly for the first time as respects brick masonry the fact that the strength of a masonry wall is a definite function of the strength of the individual units. In the generally accepted range of brick strengths and with cement-lime mortar, this was about 25% to 30%. With good workmanship and cement mortar as high as 43% of the unit brick strength could be developed in wall strength. Even with poor workmanship and cement-lime mortar, over 20% of the brick strength was obtained. The full sized walls of 6 by 9 feet were only about 78% as strong per square inch as the 18 by 34 inch wallettes. In a specific case a 3320-pound brick gave a wallette strength of 1344 pounds per square inch and a full sized wall strength of 1145 pounds per square inch.

These results parallel previously ascertained facts regarding other structural materials. It is well known that the strength of a column is proportional to its ratio of length to its least dimension or thickness.

Thus a wall being considered as a series of adjacent columns will have a strength factor in the same proportion of its height to its least dimension. The McBurney tests just referred to have definitely established this ratio for brick masonry, expressed in a ratio of wall strength to brick strength. Full sized concrete walls have not as yet been tested in a similar manner, but since concrete for buildings is universally designed to be a 2000 pound concrete as measured by a 6 by 12-inch cylinder, it would follow from the McBurney tests that a concrete wall should be considered as a wall composed of units of 2000 pound concrete, and it would thus follow that under similar conditions the strength of a concrete wall would bear a ratio of wall strength to unit strength, the unit being 2000 pound concrete. This is borne out by a recent series of tests made by the Bureau of Standards upon very large concrete cylinders running as high as 36 inch in diameter by 72 inches in length, the result of which are published in the technical news bulletins Nos. 129, 133, 135. From the latter paper we quote the following: "Comparisons of test results readily show that in general throughout the tests for any batch, the larger the diameter of the cylinder, the lower the strength."

The compressive strengths shown by McBurney were all developed on walls laid up in American bond with a header course every 6th course of brick. Previous authoritative tests by the Watertown Arsenal showed that greatly increased strength could be obtained by varying this header course. For example, an increase of 57% was obtained with a header course every 3rd course. While this subject has not yet been fully investigated, it is clear that the McBurney figures are at least ultra-conservative.

While the question naturally arises in the mind of a California designer as to the extent to which brick supplied for construction in this state will show values similar to those developed by the Bureau of Standards, tests have been made of the brick in all the important construction areas. In San Francisco the average common brick has a compressive strength of upwards of 2500 pounds. This same figure is common for the Los Angeles area. A series of tests made by the Clay Products Institute of California, showing an average of 2850 pounds of brick from a large number of plants. The range of brick in these tests was from 1967 to 5832 pounds. Tests in San Diego indicated a range from 2100 to 4400 pounds, while the average brick sold in Fresno is approximately 2400 pounds. In any of these markets brick of a minimum unit strength can be obtained for use at any construction where high stresses are likely to develop. There are, of course, sections in the state where brick of poor quality has been made and sold, but it is safe to say that over 90% of the construction is carried out with brick averaging over 2500 pounds per square inch.

Shear

The discussions in the earlier chapters have shown that the earthquake forces must be resisted by the strength of the wall in shear as well as in compression. It therefore becomes important to determine what may be expected from brick masonry in direct shear. The tests of the Bureau of Standards showed that the shearing strength of individual brick is at least 70% of their compressive strength.

The Bureau of Standards tests also established the fact that the compressive strength of a brick wall is from 20 to 30 per cent of the strength of the individual brick, variation depending upon the mortar and workmanship. The mortar, however, being in general in proportion of approximately one part of cement, one part of lime and six parts of sand. On this basis the shearing strength of the wall can safely be assumed to bear the same relation to the shearing strength of the brick as the compressive strength of the wall to the compressive strength of the brick. Thus the shearing strength of the wall can be expressed by the formula:

$$\frac{\text{Wall strength}}{\text{Unit strength}} \times \frac{\text{Unit shear strength}}{\text{Unit strength}} \times \text{Brick strength}$$

Using the Bureau of Standards values of .20 for the first ratio and .70 for the second ratio and assuming a 2500 pound brick the shearing strength of the wall (Sw) would be:

$$Sw = .20 \times .70 \times 2500 = 350 \text{ lbs. per sq. in.}$$

This would therefore indicate a wall shearing strength with cement-lime mortar and a 2500 pound brick of 350 pounds per square inch. Confirmation of this determination is readily obtained. It has been established that the following relationship exists between compressive strength and shearing strength, viz.:

$$Ss = \tfrac{1}{2} Sc \times Cot\, \Theta$$
Where Ss equals shearing strength and,
Sc equals compressive strength and,
Theta equals the angle of rupture.

(See Johnson's Mechanics of Materials: Theory and Experiments on Laws of Crushing Strength of Short Prisms, by Bouton; Washington University.)

In the average panel 9 feet high by 18 feet long this angle is 27 degrees with the horizontal. The cotangent of 27 degrees equals 1.96. Solving this formula for a wall constructed of brick having a unit compressive strength of 2500 pounds and a resulting wall strength of 500 pounds gives a value of the shear strength of the wall of about 490 pounds per square inch.

That these figures are consistent with the determination of shearing strength on other material may be evident by reference to the authorities quoted by Mr. Butts on page 76, in which it is shown that concrete develops a strength in direct shear of from 500 to 1200 pounds. Equally high values for shear in brick masonry can be readily obtained by a higher strength brick and by selected mortar and workmanship.

It is important to observe that the shearing value of a good brick wall is greater in pounds per square inch than the shearing value per square inch obtained by testing 2 or 3 units having 1 or 2 mortar joints

because of the greater mechanical bond existing in the mortar joints in the wall.

If a wall was constructed of 2 huge bricks with only 1 mortar joint along the angle of rupture, then of course there would be a direct shearing action upon a single smooth mortar joint. But in the brick wall the vertical mortar joints both crosswise and longitudinally with the wall, and also the ends of the bricks form a continuous mechanical key for the horizontal and vertical mortar joints, which all works together to greatly increase the shearing resistance of the whole wall upon the diagonal line which passes through both the bricks and the vertical and horizontal joints.

The mechanical bond of the vertical joints and of the ends of the bricks is recognized as an important factor also when the shearing force or theoretical line of rupture is directly horizontal, as it would be next to impossible to cause a brick wall to shear off on a single horizontal joint, as the shearing force would be absorbed through the mechanical bonds and diverted into compression components which, of course, would withstand a greater shearing stress than in a case as above suggested wherein the wall might be composed of 2 large bricks with a single horizontal joint, which might be erroneously considered the theoretical horizontal shearing value of a masonry wall.

It is to be particularly noted that the shear thus referred to is *direct* shear as discussed by Mr. Butts in the chapter on the "Effects of Earthquakes on Structures" and is not shear developed by bending. In the analysis of the assumed structure presented by Mr. Butts, he showed that the shear per square inch on the first floor of the 13 story building having the dimensions given, would be not more than 182 pounds per square inch. This was shown to be an extreme case, both in the proportions of the building and because the walls are counted on to supply *all* the resistance to earthquake stress, and these far exceed the usual conditions. It would thus appear from the known strength of brick masonry in shear, as shown above, that a 12 inch brick filler wall would have adequate strength in shear to provide the necessary rigidity to withstand an earthquake of the intensity of 1/10 g (3.2 feet per second per second acceleration). Indeed for most buildings an 8 inch brick wall would suffice, while on the other hand, there might be exceptional cases of design where greater than 12 inches would be required.

Mr. Butts has assumed that the filler walls would be used to supply the necessary stiffening for the building. Under some conditions of design even with the building rigid throughout, the stiffness is provided in the frame by additional steel in the beams, special connections, knee braces, or other means. In these conditions, of course, a filler wall is not called upon for greater strength in either compression or shear than will support its own weight. From an earthquake standpoint its chief requirement is that it shall not be thrown out of the frame. Experience shows, however, that this contingency is remote, as is also demonstrated by Mr. Butts on page 66, through a mathe-matical analysis of the forces. Nevertheless the wall should be definitely anchored to the columns and spandrel beams. Where the columns are encased in concrete this anchorage can readily be provided by dovetail anchors in the columns or by leaving a grooved recess the full height of the column into which the brick work can be keyed.

If the walls are to act as bracing material it is highly important that they should be built up solidly and closely against the framing on both sides and the top so as to prevent movement of the framing before being braced by the filler wall which would set up impact upon the wall and might cause stresses in excess of those shown by Mr. Butts. The bricks should be laid tightly against the upper spandrel beams fitting snugly into the mortar so that there will be no space between the top of the wall and the framing above.

Other Filler Wall Construction

Brick for filler wall construction therefore, if used with intelligent consideration of the forces to be met with, provides a very satisfactory earthquake resistant material. Is it the most satisfactory? To answer this question it becomes necessary to consider the possible alternative methods of building filler walls. The use of hollow clay tile will be given attention in our next chapter and we shall therefore consider briefly in this chapter the relative merits of reinforced concrete.

Here we find many misconceptions to be current in the minds of engineers. The argument in favor of concrete has generally centered around some such statement as "since the earthquake force sets up alternate tension and compression on diagonals, any material which cannot withstand such tension must crack." Now we have already seen (page 75) that if the wall material is adequate in compression and shear no tension member need be provided. But even taking no account of this fact can we assume that if tension does exist, concrete will render any better account of itself than brick? It is admitted that plain concrete is not a tension material and tension can only be allowed in reinforced concrete to the extent justified by the reinforcing. But this is not all. The reinforcing must be placed so that the steel will take the tension. As has been pointed out, the earthquake force produces tension *along the diagonal*. If diagonal reinforcing is possible in a filler wall and if the reinforcing rods or bars are so placed with reference to the frame that they will develop such tension, then such a wall can be designed to supply whatever amount of bracing is necessary by increasing the diagonal stee. But this is *not* the type of reinforcing generally used in filler walls and it is in most cases impossible because of the necessity of putting in a large number of windows which preclude the use of the diagonal bracing.

The customary practice of placing vertical and horizontal reinforcing bars does not provide this diagonal tensile strength and moreover the bars are usually not tied in definitely to the framing. The rein-

forcement then may be useful against transverse force such as wind pressure, but it does not increase the strength of the wall in shear.

At its best a concrete wall without construction joints will have a high value in direct shear. The difficulties with the use of the material are the many variables which prevent the practical construction from realizing the ideal. Some of these may be indicated.

Variability of Concrete Materials

Variation in the quality of sand is one of the most important causes for differences in resulting concrete. An increase in the silt loam or fines, in the sand from 4% of the total to 7% has been known to decrease the strength of the resulting concrete from 3500 pounds to 2000 pounds and in general will affect it as much as 30%. Similarly the presence of mica in the sand will reduce the strength since this material does not permit adequate adhesion of the cement. Sands vary greatly in hardness and in uniform grading from fine to coarse which again produce their results upon the concrete. Moreover it is impossible even for an expert to determine the quality of sand by inspection unless aided by detailed analysis. Thus this ingredient of concrete alone, extremely difficult to control, may greatly change the resulting product.

The rock aggregate in concrete has the same variables as sand with greater stress being laid upon the strength of the rock. While crushed rock is considered safer than screened gravel, it is well known that feldspar and decomposed granite will hang together through a rock crusher and find its way into concrete. The rock is the heart of the concrete, as the cement is used to bind the rock together. The physical strengths of rocks vary with the geography of the world, as has been demonstrated by innumerable tests where the rock was the only variable. Concrete is no better than its rock aggregate, and the extreme difficulty of judging the strength of a rock from its outside appearance is obvious. This is in sharp contrast with the aggregate of a brick wall, in which the brick unit is the aggregate or rock and in which the units are manufactured to a known and constant strength. Of late the so-called "water-cement" ratio has been shown to be the greatest of all the variables in the strength of concrete. As little as a pint of water in a batch will affect the strength of the concrete. In order to maintain a constant water cement ratio it would be necessary to make a slump test on nearly every batch of concrete as it leaves the mixer, as the water content of the sand and of the rock varies with each load delivered to the job together with atmospheric conditions from day to day. Reinforced concrete is designed by highly skilled minds which then must entrust the execution of their plans to the thoughtless laborer who has to contend with the wide variables mentioned, of which he is almost wholly ignorant.

Placing Concrete

With a given strength of concrete its value depends upon its placement in the structure. The placing of concrete is purely individual and the individual is usually an unskilled laborer. The foreman, as we all know, is concerned chiefly and almost wholly with the constant flow of the concrete or the quantity per day and the inspector can not watch 10, 20 or 40 individuals at once. The object of the concrete laborer is to get the concrete in place or out of sight as quick as possible. The matter of honey comb, rock pockets, grout leakage, disturbance of reinforcing bars partially set and the disturbance of their bond are of no concern to a laborer, but are variables vitally affecting the strength of the concrete. It may be appropriate to point out that while concrete walls are largely constructed out of sight in forms, brick walls are constantly exposed to the eye of foreman or inspector who could at once detect voids or lack of mortar. Bricks are laid by skilled workers, not by ordinary labor. Concrete forms are seldom constructed to carry the wet mass of a concrete wall for its full height at one pouring, it being the recognized custom to pour walls in a series of layers of from 2 to 4 feet or so at intervals of considerable time. Upon the top of each layer there is an accumulation of laitance which is an inert material forming a cleavage joint not capable of any bond. Such a joint or joints in a wall form definite weak danger planes in a wall subjected to an earthquake shock, as on these planes there is little or no adhesive bond in shear. In this same connection columns are poured up to the beam bottoms or column tops and allowed to set for 2 hours or so for shrinkage and it is extremely difficult, if not impossible, to remove all of the laitance from a column top before filling the column through the floor system. These laitance joints on columns are readily visible in most reinforced concrete structures. This point is recognized as a vital one in an earthquake disturbance and experience has shown that it is at these points that failures occur.

The importance of having the filler walls directly against the frame where such wall is intended to brace the structure, has been shown above. Methods of placing concrete make it practically impossible to attain this desired end because in pouring the concrete in the top of the wall and around the spandrel beam which is at the floor line, there is not sufficient head or pressure on top of the wall to force the concrete up under the bottom of the spandrel beam and even if it reaches this position initially the shrinkage during the set of the concrete leaves a void which makes possible movement of the frame before the bracing effect of the wall is felt and develops impact upon the wall itself, thus greatly increasing the stress upon the wall.

Changes After Placing

Concrete is not a finished material even when placed. The best concrete may still be spoiled if not surrounded by the right conditions for curing during the next few weeks or months. In California especially, where hot, dry weather is frequently the rule, improper curing is the cause of much crazed or cracked concrete.

There is danger as well as fallacy in the current opinion among many architects and engineers that if concrete survives the stresses of the construction period its security will continue to increase. Chemical and physical changes are constantly going on in concrete and are attracting the attention of outstanding students. In regard to the former in the A. S. T. M. Proceedings, Volume 23, Part II, 1923, there is a discussion of "What Properties of and Methods of Making Concrete Require Further Investigation." Mr. Frank C. Wight, the managing editor of the Engineering News-Record, on page 158 of this volume says:

"Concrete is a chemical combination subject to all the variations in structure that that implies. To be sure, both steel and wood are chemical combinations, but the reactions in the former are so slow as to be negligible, and in the latter obvious and well understood. Concrete, on the other hand, is an obscure compound, the chemical nature of which is debatable, and whose reactions are neither understood nor measurable. Nevertheless, these reactions take place and their occurrence continues over an unknown length of time. It is not beyond the possibility of belief, for instance, that there is some truth in the claim that there is a critical age in concrete—a period somewhere between 10 and 15 years, maybe more, maybe less—when there occurs a subtle change in its nature . . . Only those blinded by prejudice can fail to admit that a great deal of existing concrete falls far short of the expectations of its designers in appearance and in structure. Most of this concrete was satisfactory when the forms were first removed. It is probable that when the builders left the job they were convinced that they had turned over a structure that would continue in the acceptable condition they left it. For some reason or other there has been a change with the passing of time—not change due to unexpected or undue load or wear, but change due to the natural exposure of the cements and natural reactions in concrete itself."

Other changes partly chemical and partly physical are constantly taking place in concrete. Some of these may be described as follows.

A steel frame expands and contracts with the varying temperatures of weather, very slightly when measured in small distances, but quite perceptible when actually measured on a large structure. The case of the steel frame of the Los Angeles City Hall may be cited. The steel was fabricated in the summer season. It was found in the winter season that the longitudinal and transverse center lines of the building were dead true with the permanent outside witness points, but that the column centers when measured off the main center lines, showed a uniform shortage in distances from the true exact distances which were still maintained at the bases. The total shortage in the width of 274 feet was ¾-inch, and in the length of 476 feet was 1½". While it is true that the steel frame was not encased or protected and took a free shrinkage, it is also true that the total temperature differences were very moderate. A great many floor systems in skeleton frame buildings are constructed of some system of reinforced concrete, which is cured for many weeks or months before the finished floors are laid. We know that the interior of a building is more uniform in temperature than the exterior walls, and yet the floors continue to shrink or expand and crack, either from the nature of

the concrete or from the changing of the frame or both. We have all seen these cracks in floors. But the exterior walls are subjected to greater temperature changes by far than are the floor systems. What then must be their condition? The filler walls being poured about 8" thick and containing reinforcing bars, anchors, etc., must be, or at least are, poured quite wet in order to flow into and fill the forms, under window sills, lintels, etc., and thus the water ratio is bound to be such as will cause excess initial physical shrinkage in the wall and also excess shrinkage from subsequent chemical reactions and setting. When concrete shrinks it induces tensile stresses within itself, which are at a maximum at the point of bond with the steel and a minimum at the exterior surface. The matter of volume changes and initial stresses set up in concrete have been the subject of considerable study and actual measured tests and proofs by many eminent authorities whose published findings point out that there are initial stresses set up in reinforced concrete. In reinforced concrete the shrinkage is prevented somewhat in the interior by the bond to the steel, but on the exterior surface, and also at points between reinforcings the concrete has minute shrinkage cracks.

Now when the frame expands or contracts the fine cracks in the walls naturally form cleavage planes, as they have no cohesiveness, and in time the cracks become larger and eventually of some concern. Consider the value in "tension" of such a wall subjected to the impact of an earthquake. In "compression" these hair cracks will close up and do some work, but such a wall could not possibly absorb or transmit any "tensile" stresses at all, especially on a diagonal line with reinforcings running vertically and horizontally at the customary spacings.

In order for the steel in the bottom of a beam or girder to "act" or supply its 16,000 lbs. per sq. inch resistance, the concrete in the bottom of the beam must crack or "give" even to a very slight degree. Consider the spandrel beams in a reinforced concrete skeleton frame with reinforced concrete filler walls, all poured more or less monolithically. When the load comes to the top story spandrel beam it must receive the load— it should carry the load to its supporting columns. But the steel in the beam must act in tension and the beam must take *some* deflection in order to stress the reinforcing steel and distribute the load. But the concrete filler wall poured monolithically with it, will not allow the beam to deflect, at all, but takes the load into the wall.

The wall cannot deflect vertically because it is resting upon the beam below, which in turn is receiving its load from its floor and again cannot deflect for the same reason as above, and thus the spandrel and wall loads are accumulated and sent on down to the lowest wall or beam.

The accumulation of loads carried down through the concrete filler walls, are either partially distributed to the columns or the walls crack, or else if the walls are strong enough they withstand all they can until

some unusual jar, impact or vibration causes them to fail—crack.

The steel spandrels of the steel frame building must also deflect. It is practically impossible to get concrete to actually contact the full under side of the bottom of steel beams as hereinbefore discussed, but the under sides of the top flanges are more apt to be fairly filled because of the angle or slope of the insides of the flanges. This again will prevent the steel beam from deflecting and the same accumulation of loads will develop.

This is not blind theory. Many tall buildings on the Pacific Coast and in the central and eastern cities having cracked window sills and heads, mullions, etc., clearly show the fallacy of not providing for spandrel beam deflection.

If the building is not tall enough to cause the lower walls to crack, the walls may nevertheless be stressed to such a point that an earthquake shock would cause an over stress and thus rupture the building.

Brick work, with a *cement-lime* mortar is slower setting than concrete because of the retarding action of the lime in the mortar and when the load of the "green" or "fresh" brick wall in the story above is received, the spandrel beam whether of reinforced concrete or steel, then takes its deflection, and thus compresses the fresh mortar joints of the brickwork immediately below the beam and the beam loads are then properly delivered to the columns at *each* floor. The mix for mortar is usually 1:2:6, as against a 1:2 or 1:2½ for concrete. The masonry mortar is thus very slightly compressible and minutely elastic, enough so that when taken into hundreds of joints it can be minutely distorted without rupture or failure, while the concrete is a brittle material which will, and does, rupture more readily upon receiving a shock or impact.

In this same connection it should be observed that the general custom of using a "rich" mortar, usually a 1:3, for facings such as terra cotta or face brick and then using a mortar of 1:2:6 or poorer for backing, produces the same effect upon the facing as upon the monolithic concrete filler wall, in that the backing *does* compress with the deflection of the spandrel beam, while the facing is set as a hard unit like a tall sheet of plate glass and then strained from behind, at each floor and does *not* deflect. Of course it cracks. In this case, however, the damage is only upon the facing.

Authorities Discuss Concrete

A distinguished authority, George F. Swain, Professor of Civil Engineering at Harvard University, past president of the American Society of Civil Engineers and a member of many other technical societies, in an address before the American Institute of Steel Construction, October, 1926, stated:—"Concrete is not a finished product. It is made on the job. The engineer must not only design the structure, he must also design the material. At first concrete was proportioned quite arbitrarily and it was thought that the only ingredient to be carefully controlled was the cement. Then it was recognized that the sand and the stone were important. Finally it has come to be generally believed that the water is the most important of all, and we are told that the strength depends almost entirely upon the ratio of water to cement which should be as small as possible so long as workable mixtures are obtained and that the grading of the aggregate which produces the strongest concrete is not that giving the maximum density.

"Another most important characteristic of concrete is that it changes after it is placed beyond recall in the structure. In this respect it differs radically from other structural materials. The real manufacture of the material begins then when it begins to set and harden. The phenomena of setting and hardening are not yet fully understood . . . All concrete unless waterproofed is more or less porous and will swell and shrink when wet and when dried with varying moisture in the atmosphere . . . Its ultimate condition depends much more upon the curing during the setting and the amount of water given it. It expands and contracts subsequently as it is wetted and dried. Is it not obvious that there must be considerable uncertainty in the product? Modern methods of field control have made it easier to secure uniformity and good quality, but great lack of uniformity still exists. If a pint of water in a batch makes so much difference, how can we expect uniformity with ordinary workmen, varying weather and varying moisture in the material?

" . . . Further it is to be observed that before a beam can act according to the usual theory with the rods stressed to say 16,000 pounds, the concrete must crack. This is the only structure composed of 2 materials in which the accepted theory presupposes the actual rupture of one of the materials. This certainly does not look very good."

Further explanation of the changing character of concrete is brought out by Raymond E. Davis, Professor of Civil Engineering, University of California, Berkeley, one of the most eminent authorities on the materials of construction, as a result of his studies and tests extending over a period of many years. In a paper presented by Professor Davis at the International Congress for Testing Materials at Amsterdam, September, 1927, and read to the Congress in the absence of the author by Professor D. A. Abrams, perhaps the leading concrete authority in the world, Professor Davis states:

"While it has been recognized in a general way that the permanency of a concrete structure is dependent upon more than mere strength, surprisingly little quantitative data are available concerning certain other properties of concrete which are often of equal or greater importance in structural design. Chief among these is the property possessed by all portland cement mortars and concretes of changing their volume not only during the early period of the hardening process, but thereafter, as variations in moisture conditions occur. Just as a clay shrinks when allowed to dry and swells when again moistened, so does concrete undergo changes and, except it be under water or

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be maintained in an atmosphere of constant humidity, *these volumetric variations are continuously in process, certainly for a long period of time and perhaps throughout the life of the material.*

"The reasons underlying these volumetric changes due to causes other than variation in temperature are not altogether clear. It is impossible to say where the effect of changes from one source leaves off and that from another source begins; but perhaps both chemical and physical variations are always involved. The chemist is likely to ascribe this action to shrinking and swelling of the colloids of the cement; the physicist might ascribe it to variations in capillary tension within the pore space of the mass. From whatever source, it is a factor to be reckoned with in engineering design, *and concrete structures, except those under water, may hardly be regarded as truly permanent unless so designed that the volumetric changes may take place without producing excessive stress.* Within the past few years there have been numerous examples of the slow failure of concrete which could only be ascribed to this cause, and there can be little doubt but *that this property, of shrinking when allowed to dry and expanding when wet, is the greatest single factor tending to disintegrate our concrete structures.* So while concrete is an excellent material for many purposes, and is likely to be used even more in the future than in the past, it unfortunately possesses the property of variability of volume which, though generally small, cannot always be ignored, but must be quantitatively considered unless our unhappy experiences are to continue."

The quotations from these eminent authorities substantiate the experience of every careful observer that the average concrete exposed to the elements on the exterior of a building cracks over a period of time as a result of the volumetric changes taking place within itself. The fact that a reinforced concrete exterior wall has a definite strength and soundness shortly after construction, gives no assurance that after the changes which are constantly taking place in it have occurred that it will retain either. Clearly the important thing with respect to filler wall construction in our major buildings is that it be of permanent character.

Evidences of the effects discussed in the last few pages can be found on every hand. One of the largest and most prominent public buildings in Los Angeles designed by an eminent architect and constructed by a very reputable contractor, has developed within 3 years very pronounced cracks evidently due to the expansion or contraction of the material. On the side of the building that is exposed to the sun as well as moisture and therefore experiences the most extreme variations in temperature, these cracks going completely through the wall, and in some cases, into the beams or columns, are evidenced every 30 or 40 feet. Some of them are an eighth of an inch wide. This condition is not by any means unique but is found in the walls of many of the large concrete buildings in California when examined.

Even a small crack will allow the moisture to attack the reinforcing steel and thereafter it is only a matter of time until the critical condition is reached. Our skeleton frame buildings are generally intended for a life of at least 40 or 50 years, and if a material is not dependable for this length of time it will have very definite limitations for use in filler wall construction.

We have seen in our descriptive chapters how many and serious were the failures in reinforced concrete buildings in Japan and Santa Barbara. Yet in neither of these cities nor in San Francisco were there any concrete structures of any considerable age at the time of the earthquake. In Japan particularly all of the reinforced concrete construction was of very recent date, none of it having reached the 15 year period indicated by Mr. Wight as critical and most of it not having passed the 2 to 5 year period, which is another time of internal change.

From what we have pointed out it is evident that the most severe dangers to concrete occur where it is exposed to the outside weather conditions and therefore subject to alternate wetting and drying, heat and cold. These objections are therefore particularly applicable to filler wall construction in skeleton frame buildings and are not as serious as affects concrete beams or columns in the interior of structures.

That the engineering world is becoming appreciative of the situation we have described is evident from the editorial in the Engineering News Record, January 17, 1929.

"A Necessary First Step"

"Two concrete structures built a few years ago under similar conditions were recently examined. Both had been built by experienced engineers and capable contractors, under approved specifications. Yet one of them now looks clean and new, while the other has a decaying surface. Why? An answer has not been found, but *we believe the question is as serious as any now before the engineering world.* It might be thought that the one contractor was less skilled than the other. But some miles away there is a third structure, involving only one contractor and one set of workmen, which shows parts that are going to pieces and other parts that have remained sound; it is hard to believe that this contractor was competent at one moment and incompetent at another. Something else must have been at fault. We happen to know personally that in all three cases the engineers had inspected the work and found it to comply with their specifications. If, then, a specification carried out with at least average skill and faithfulness does not assure the expected results, we are forced to ask whether the specification does in fact specify effectively. Puzzles of the kind presented by the three structures mentioned are neither new nor rare, *but lately they have been getting insistently obtrusive. If they continue, what of the future?* The past year or two seems to have done little toward their solution. The current year ought to do better. It should at least see a determined attack on the problem—its difficulties analyzed and discussed without reserve."

Relative Costs of Repairs of Earthquake Damage

This subject cannot be left without some consideration of the relative cost of repair, since earthquake damage is measured by the cost of repair or replacement.

A patch is not a repair.

To repair is to restore the original value, use and purpose of a material so that it can be relied upon to fully perform its duty to the same extent as the original before it was damaged.

The simplicity with which a brick wall can be repaired by cutting out the damaged or broken bricks or mortar and by replacing them by shoving new bricks and mortar into place is too well established to require further comment.

Such is not the case with reinforced concrete, however, and it therefore is fitting to refer to some authorities who are well conversant with the problems of repairing this material. The problem being not only one of getting the repair material into place, but also to make it actually *bond* on to the old work. Bricks always retain their "suction" for mortar. There is little or no suction to concrete.

Following is an extract from an editorial which appeared in "Railway Age" October 16, 1926:

"Bonding New Concrete to Old"

"The question of securing a proper bond between concrete which has attained its final set and green concrete deposited thereon has engaged engineering thought for many years. It is vitally essential that the contact surface of the old concrete to which the new material is to be applied be *scrupulously* clean, and also that it be saturated with water as a preliminary to the placement of the repair concrete. A further object in the application of water is to *expand the old concrete* to a degree which will co-ordinate it with the green mixture.

"The initial step in the prosecution of repairs is the removal of the defective concrete, and the exposure of a clean surface of sound material. This should be carried to a sufficient depth to insure the detachment of all particles of disintegrated or impaired concrete and provide a surface of clean, sound material. Reinforcement—(must be clean, uninjured, etc.) . . .

* * * *

While the establishment of any definite formula for the proportionment of repair concrete is difficult, it is desirable that this be a fairly rich mixture, at least that undue porosity be avoided. As trouble has sometimes arisen due to cleavage resulting from the contact of two masses of concrete of *differently proportioned* mixes, the relative quantities of ingredients in the old concrete should be considered and the new concrete should not be permitted to depart so widely from the ratio as to create the possibility of future difficulty from the source mentioned."

Following is an extract from an article by Mr. Henry D. Dewell which appeared in Engineering News Record July 9, 1925:

"The method of repair in those reinforced-concrete structures which have suffered but slight damage—for example, when a few of the first floor beams—is a problem which warrants careful consideration. Obviously all of the damaged concrete must be removed. If the reinforcement has not been distorted or bent out of place it is possible that after the damaged concrete has been removed the column may be built up to its original shape with rich con-

crete and that it will then be able to function as well as before. *After that had been done, however, the writer feels that the finished product would not be as satisfactory as new construction.* In this respect the structural-steel frame has a great advantage, because where a structural-steel frame is damaged at the connection by shearing of the rivets, a repair of this damage would restore the connection with certainty to original strength."

We have seen that filler walls resist earthquake forces in compression and shear. When a filler wall is cracked or damaged it must be repaired with both of these elements in mind, shear as well as compression. With shear is the element of adhesion and cohesion or *bond*. To *bond* new concrete to old concrete and form a repair which is 100% perfect in performance is costly as well as difficult, but a brick is always a brick and cement-lime mortar always bonds to a brick whether a new brick or old brick.

Reinforced concrete is a *mass* material and is economical only as such when it can be poured into place. Brick masonry is a *unit* material and is economical as such whether in small or large quantities.

At least 95% of all small masonry jobs such as filling in openings, etc., both in brick and concrete walls are done in brick masonry rather than in concrete because of the greater economy of the brickwork. So where masonry work is complicated and is relatively difficult to perform and yet where maximum strength is required such as underpinning, piers, fireplaces, chimneys, etc., brick masonry is invariably used because of its adaptability, positive strength and economy. It follows that cubic foot for cubic foot brick work can be repaired, restored more economically than can reinforced concrete when done by hand by mechanics and hand tools.

The *value* of a repair is the important thing: that is, whether after all it is a *repair* or a good looking patch. Accordingly, while it is impossible to inspect and know the value of the "inside" of reinforced concrete or of the inside of a concrete repair, it is always possible to inspect and know the value of the inside of a brick wall and of a brick and mortar repair.

The cost of the masonry repairs on each one of 4 institutional buildings in Santa Barbara (after the earthquake) containing a total of about 1,200,000 common bricks and about 150,000 hollow tile, was less than $500.00 per building.

In Tokyo, 1923, out of 18 buildings owned by one corporation, and having steel frames with brick walls, costing a total of $30,074,000, the cost of the repairs from earthquake damage was $750,000, which was spent on 6 of these buildings whose cost totaled $15,321,000, the other 12 being undamaged. This repair cost can be otherwise expressed as 4.9% of the cost of the buildings affected or as 2.5% of the cost of all the 18 buildings owned by that company.

Brick work has another great advantage over monolithic construction when being repaired or altered, in that there is invariably a large percentage of salvage in the old bricks themselves which continue to retain their original qualities of strength, etc., and which

remain on the job and are re-used, while concrete cuttings are entirely waste and must be hauled away and all new materials used, which may *differ widely in quality and performance* from the original mass.

This point is further evidenced in street widening programs wherein buildings having such walls and facings have been cut back and then the original bricks and terra cotta have been re-used, whereby the structure has exactly retained its former appearance and strength and at a minimum of cost.

Summary
In this chapter we have seen that the official tests of the Bureau of Standards have shown a uniformity and dependable strength in brick masonry; that brick throughout the state of California is generally being delivered of high quality.

It has further been shown that a brick filler wall will have a value in direct shear of from 350 to 500 pounds, whereas the highest calculated shear in similar walls under severe earthquakes is 182 pounds, hence that a brick wall, if of adequate thickness, provides ample strength to brace a skeleton frame structure. It has been shown that the chief competitor of brick walls, reinforced concrete, has many variables in composition, in placing, and particularly in the changes which occur after the material is in place. Also that the reinforcing in such walls is not generally placed to supply diagonal tensile strength. Eminent authorities have been quoted to indicate the changing character of concrete as indicative of its limitations for exterior wall material. Where concrete has cracked its earthquake resistance is materially decreased. Finally we have seen that in case of damage the brick wall can be quickly and effectively restored to its original condition, whereas the reinforced concrete wall can only be repaired at greater expense and with the probability of not restoring its original strength.

The discussion has been limited strictly to problems arising out of earthquake conditions and has taken no account of the other advantages of unit masonry. The effectiveness, however, of the brick filler wall in the skeleton frame building would seem to have been demonstrated.

The Brick Bearing Wall Building

The Purpose of This Discussion

We have seen in recording the effect of earthquakes upon brick bearing-wall buildings in San Francisco, Japan and Santa Barbara, that, while there were some slightly damaged, others heavily damaged and some utterly destroyed, there was, on the other hand, a substantial proportion of such buildings in the same area which successfully withstood these disturbances unscathed. To a thoughtful mind the question is immediately raised as to what is the real basic reason for this.

In order properly to understand these different performances of brick bearing-wall buildings, it is necessary to analyze the stresses set up in the building during such a disturbance and also to study the design of the building itself to determine the factors vital to the resistance of the structure.

Such an analysis and study will also disclose the critical points which must be watched in a building, how the various elements of design and construction affect and contribute to its resistance and stability as a whole, and how an earthquake hazard may be largely eliminated by a reasonable consideration of these elements.

An analysis of the stresses induced by the earthquake has been given in Chapters II and VI of this presentation. It will therefore be unnecessary to discuss the nature of these stresses here except to repeat that as shown in those chapters, the only earthquake forces which need be considered in detail are the short surface and direct horizontal waves.

The Function of Structural Brick Masonry

Brick masonry itself is a rigid combination, therefore its primary structural function is to resist rupture by compression and shear, and secondarily to resist bending.

In Chapter VII is shown how the unit values of compression, shear and tension depend entirely upon the "designer" of the masonry; that is, brick masonry can have and does have the strength which the builder demands, simply by determining its composition and providing competent inspection to insure its fulfillment, the same as he does for steel, reinforced concrete, timber or any other structural materials. It is shown that good brick masonry develops at least 700 lbs. sq. in., in compression (U. S. Bureau of Standards) 350 lbs. sq. in., in shear and should develop around 100 lbs. sq. in., in tension with a 1:1:6 cement lime mortar (from University of Washington tests); based upon a brick of only 2500 lbs. sq. in., compressive strength. With a 4000 lbs. brick (a common

average) the values would be proportionately higher. (U. S. Bureau of Standards report, 1928.)

The responsibility for the strength of the masonry in actual pounds per sq. inch seems to rest between the owner and the builder. The owner cannot expect to get high value in construction unless he pays for it and conversely if the work is skimped in every way it is likely to be weak. When a price is set on steel, copper, glass, paint or any other standard commodity, the buyer pays for the inspection of the material *indirectly,* but it is *inspected* to a certain quality nevertheless and he gets the quality for which he pays. In combining brick, cement, lime, sand and water through the medium of only one principal variable—*workmanship,* to produce a mass of structural value and structural responsibility, it is only reasonable to require that all structural masonry be rigidly inspected. The values used in this discussion are those based upon inspected, average good workmanship, and upon a low average common brick of only 2500 lbs. strength.

The actual function of the brick masonry during an earthquake is to resist by compression and shear the forces delivered and distributed to it by and from the interior structure of the building.

The Function of the Interior Construction

As stated before, the earthquake forces are delivered to all parts of the building simultaneously and not just to the outside walls; however, the interior forces are ultimately delivered to the outside walls. The masonry is rigid, but the interior wood construction is more liable to yield, hence it is of utmost importance to observe these points: that the interior construction is made rigid in itself by effective bracing in the partitions and floors, and in all roof framing, both horizontal and vertical in the longitudinal direction to keep this framing from distorting; and *all* interior work is *tied together* such as actual joist *splicing* over bearings, partitions thoroughly tied to the floors and ceilings, sub-floors thoroughly nailed to joists, joists thoroughly bridged in spans and solid bridged at supports, etc. If the interior is thus braced it will act as an entire unit without distortion, and will deliver to the masonry a *uniformly distributed* load which the masonry can be designed to safely resist.

Between the Interior and the Exterior

In any bearing wall building there are *two* elements vital to its stability. The first one is a sufficient *support* or *bearing* for the vertical loads; the second one the anchoring of the interior construction *in* to

the masonry to develop lateral stability in the structure.

The first of these must be provided if the building is to stand at all; but the second is not always provided and it is this that in a large measure accounts for some of the "queer" performances of brick buildings during a seismic disturbance, where one building will be damaged or destroyed along side of one of the same general design, materials and cost which escapes unharmed. The quake vibrates the *entire structure* and if the interior construction, partitions, etc., are not anchored rigidly *in* to the masonry, the quake sets up a vibration in these interior parts different from that of the masonry and soon hammers or batters against the masonry, a condition which is naturally destructive even if the actual distance between the interior and the masonry be only a fraction of an inch, because the weight of the entire interior is behind the blows and the impact is too much for any ordinary masonry to resist. This bombardment and movement of the *interior* construction independently of the masonry, is the real cause of perhaps 90% of the damage done to brick bearing-wall buildings during a heavy earthquake. Actual photographs taken immediately after a heavy quake bear a striking witness to this condition.

Attention is called to the Aero-photo 56, of Santa Barbara's business district adjacent to State Street, the principal business street. A study of this photo shows that the two principal contentions of this discussion are well founded upon experience. First, that the average good brick bearing-wall does not fail in shear. There is no evidence of shear failure in any of the brick walls in this area. Second, that the greatest damage done to such buildings is caused by the abnormal impact action of their non-rigid *interiors* and *not* by the failure of the brickwork to resist the direct compression, shear or bending stresses induced by the earthquake itself.

Directing attention to the right hand side of the street beginning with the first building at the lower left of the picture, it is evident that the action of the roof framing punched out the rear wall and the top of the side wall. If this was a shear or bending failure it would have broken in the lower story where such stresses are the maximum. Likewise the second and third building shows the same evidence. Notice particularly that in the third building (the one to the left of the "Fashion") the left half of the rear wall at the top has been punched out by the roof framing, not braced, while to the right of this break there are two windows over which and beyond which the brickwork is intact, which is due, undoubtedly, to the fact that there are partitions at the sides of the windows which have *braced* the roof framing and held that portion of the roof (of the same building) from punching out the top part of the masonry.

This is direct evidence of the value of rigidity in the interior construction. Notice also that the front fire wall of this same building is left intact, which is undoubtedly due to a repetition of the rear case just mentioned, partitions bracing the roof, while the Fashion building adjacent, lost both front and rear fire walls; for being an apparel store, it had no bracing partitions; the rear wall is broken down to a point even with the bottoms of the roof trusses.

The white corner building, the Pacific Southwest Bank, was unscathed; although having a very high ceiling with heavy roof trusses over, it is well tied and anchored together.

Further up the street to the right the building with the cupola tower (Boss Overalls sign) is another outstanding example of the punching out of the fire walls by the roof framing and the stability of the brick masonry to withstand the bending below, and the shear below and at the sides.

Across the street on the adjacent corner is a brick bank building with an extremely high story with Spanish roof construction which was somewhat badly cracked. This is a case where a "few pounds of iron" (anchors and reinforcings) would have saved "many pounds of gold" (for repairs.)

We might point out further evidences of the damaging effect of the movement of the interior elements of other buildings as is seen on the left hand side of State Street as well as elsewhere throughout the district, but it would be a repetition in the field already covered.

Anchoring Into Masonry

The important observation is that there is a vast difference between supporting and tying the interior *in* to the masonry, and just resting it *on* the masonry.

Girders, beams, trusses, floors and partitions should be anchored in to the masonry at all points of contact sufficiently to prevent the interior from *pulling away* from or *pushing through* the masonry, whether the masonry be direct load bearing, filler or curtain walls, or purely ornamental work. In an earthquake the interior construction must distribute the interior load equally to all masonry in contact with it, and non-*bearing* curtain walls (such as the front and rear walls and end court walls of a building which is framed crosswise) offer important resistance to the earthquake force when the loads are tied *in* to them.

To effect this rigidity while the building is being constructed is a matter so simple and inexpensive that it is difficult to understand why all builders do not provide for it. Observation of earthquake experience shows that builders have done so and their buildings stand beside those which failed because they lacked the necessary rigidity. An ounce of precaution in this matter is worth more than tons of cure.

An Analysis of a Brick-Bearing Wall Building

Other considerations, the percentage of openings, the amount of "mass" masonry in the design and the critical points in the masonry itself, can best be understood by an analysis of an actual brick bearing-wall building.

For analysis we will take an average hotel or apartment house, 4 stories high:

Fig. 1

Fig. 2

Area of front windows $= 5 \times 5 = 25$ sq. ft.
Area of side windows $= 3 \times 5 = 15$ sq. ft.
Let $Q =$ force $=$ Acceleration $\times$ Mass
$W =$ weight of mass to be taken by *one* side
$K =$ acceleration $= 0.1$ Gravity.
One floor load $= 18$ lbs. dead, 7 lbs. actual live $= 25$ lbs.
per sq. in.
$= 140,000$ lbs. each floor.
Total roof load $= 86,000$ lbs.

$$\% \text{ of openings side wall} = \frac{(14 \times 15)\ 4}{40 \times 140} = .15 = .85 \text{ solid wall.}$$

$$\% \text{ of openings front wall} = \frac{(5 \times 25)\ 4}{40 \times 150} = .125 \text{ Say } .13 = .87 \text{ solid wall.}$$

Then,

$$W = \frac{\overset{\text{(floor)}}{140,000} + \overset{\text{(2 side walls)}}{(15 \times 140 \times .85 \times 120)\ 2} + \overset{\text{(2 end walls)}}{(15 \times 50 \times .87 \times 120)\ 2} + \overset{\text{(roof)}}{85,000}}{2}$$

$$= \frac{140,000 + 430,000 + 157,000 + 85,000}{2} = 406,000 \text{ lbs., total load at third story window head line.}$$

$$KW_1 = 0.1 \times 406,000 = 40,600 \text{ lbs., estimated force of earthquake effective at this line.}$$

The net cross section area of the brick walls at the window head line of the 3rd story of the long side,

$$A = \overset{\text{(wall)}}{(140 \times 12 \times 12)} - \overset{\text{(windows)}}{14\ (36 \times 12)} = 14,200 \text{ sq. in.}$$

(In calculating shear 1.5 is the accepted constant for the impact of an earthquake force.)

$$\text{Shear} = \frac{1.5 \times \text{load}}{\text{Area}} = \frac{60,900}{14,200} = 4.3 \text{ lbs. per sq. in., which is the amount of horizontal shear in the masonry of the long side at third story window head line.}$$

At the same line on the short side:

$$W_1 = 406,000 \text{ lbs., } KW_1 = 40,600 \text{ lbs., } = 81,200 \text{ lbs., total load for all cross walks to resist.}$$

$$\text{Front and rear walls} = 2\ [\ \overset{\text{(wall)}}{(50 \times 12 \times 12)} - \overset{\text{(windows)}}{5\ (60 \times 12)}\ ] = 7200 \text{ sq. in.}$$

$$\text{Court wings} = 4\ \overset{\text{(wall)}}{(8 \times 12 \times 12)} - \overset{\text{(windows)}}{4\ (48 \times 12)} = 2300 \text{ sq. in.}$$

Total area $= 7200 + 2300 = 9500$ sq. in., all cross walls.

$$\text{Shear} = \frac{1.5L}{A} = \frac{121,800}{9,500} = 12.8 \text{ lbs. per sq. in., which is the amount of horizontal shear in the masonry of the short side, at third story window line.}$$

Similarly the stress in shear in the short sides of the *first story*, which is the greatest, would be:

$Q_3 = K\ (W_1 + W_2 + W_3) = K\ (812,000 + 491,000 + 627,000) = K\ (1,930,000)$, total load at first story window head line
$Q_3 = 193,000$ lbs., estimated force of earthquake at this line.

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Front and rear walls = 2 [(50×12×16) — 5 (60×16)]= 9600 sq. in.
Court wings = 4 (8×12×16) — 4 (48×16) = 3100 sq. in.
Total area = 9600 + 3100 = 12,700 sq. in.

$$\text{Shear} = \frac{1.5L}{\text{Area}} = \frac{289,500}{12,700} = 22.8 \text{ lbs. per sq. in.,}$$ which is the maximum amount of horizontal shear in the first story window head line.

Thus it is seen that the actual maximum shearing stress in this building due to an earthquake of 0.1g. intensity would be only 22.8 lbs. per sq. in., upon masonry, which, if laid up with solid joints, clean brick and a 1:1:6 mortar should develop at least 350 pounds in shear, or about 15 times more shear than is required.

Bending

For an analysis of the building in *bending* we have:
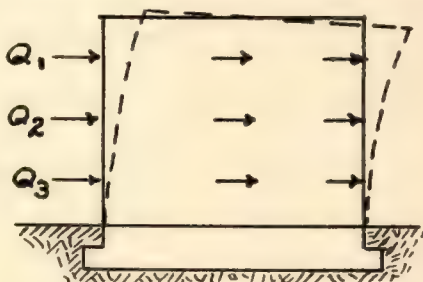
Fig. 3

$$W = Q_1 + Q_2 + Q_3$$
$$1 = \text{height} = 40 \text{ ft.}$$

$$M = \frac{Wl}{2} = \frac{SI}{C}, \ S = \frac{Wl}{\frac{2I}{C}} = \frac{1,930,000 \times 480}{\frac{4,090,000,000}{2\ 200}} = \frac{920,000,000}{40,900,000}$$

= 22.5 lbs., which is the maximum tensile stress in the extreme fibre of the brickwork.

This bending assumption is the most severe test under which a structure can be placed, and could only be possible on very spongy, filled ground during a disturbance, having an unusually short surface wave. Buildings built on such ground and subject to such a stress should be carefully analyzed for their ratio of total load to height and also their slenderness ratio, but when built on firm ground this condition of bending would not prevail.

A series of tests conducted by the Bureau of Standards at Washington, D. C. (the results of which are yet unpublished but given through the courtesy of their junior chemist, Mr. H. V. Johnson) values of adhesion of cement (1:3) mortar to *dry* bricks in direct tension of from 51.4 lbs. per sq. in. to 79.3 lbs. per sq. in. with an average among several different kinds of bricks of 66.3 lbs. per sq. in. with an increase of 70.2% in strength when bricks are wetted, which would give an average of about 112.8 lbs. per sq. in.

These bricks were previously boiled for absorption and Mr. Johnson states that the surfaces of the bricks might thus have been affected, thus a higher value might be expected; failure in nearly all of the cases was between the mortar and the face of the brick.

Another series of tests conducted at the Bureau of Standards has just been completed and the results are now available. These tests were on panels 6 feet long by 9 feet high by 8 inches thick, and also 12 inches thick, which were restrained at the top and bottom and loaded at two horizontal lines one-third distance from the ends.

Using brick having a compressive strength of 3280 pounds per square inch and a modulus of rupture of 1225 pounds per square inch, laid with one to three cement mortar, the 8 inch panels developed an ultimate fiber strength from 50 pounds to 96 pounds per square inch and the 12 inch wall developed fiber strength of 82 pounds per square inch.

The masonry work was of average workmanship and was only 60 days old as were also the adhesion tests just referred to. A longer period of time to allow the mortar to thoroughly chrystalize in the pores of the bricks would have given values much higher than those found.

The University of Washington (State) has recently conducted some tests on the adhesion of cement mortar to clay bricks and found an average tensile strength of 182.5 lbs. per sq. in.

Specimens of actual field work in Los Angeles with an approximate 1:2:6 cement-lime mortar tested at Osborne Laboratories gave an average value of 75 lbs. per sq. in.

Considering the above values and conditions it seems conservative estimate a cement lime mortar of 1:1:6 to have a tensile value *in the masonry* of about 100 lbs. per sq. in., when reasonably aged.
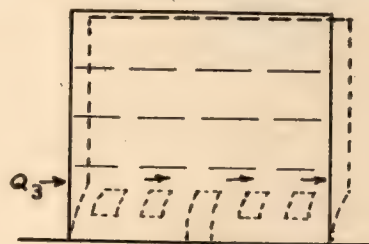
Fig. 4

First Story Bending

If the building would act as a unit above the second floor line and develop bending only in the first story (Fig. 4), then the total area of masonry to transmit the bending to the masonry walls and piers would be a summation of the cross sections

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of all cross walls with their ells at the corners, a distance out at least equal to the thickness of the wall, as at ab (Fig. 5), (beyond which point there would be bending or vertical shear in the long wall), and these ells would also act in resisting bending in the narrow wall.
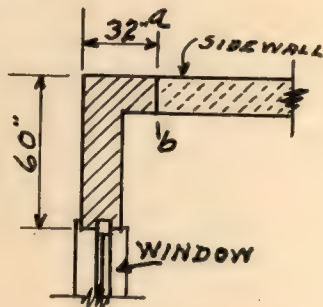
Fig. 5

Then we would have, for all the corners,

$$4 [(60 \times 16) + (16 \times 16)]$$
plus $8 [(24 \times 16) + (16 \times 16)]$
plus $6 (60 \times 16)$

equals 15,740 sq. in., total area

and $\dfrac{L}{A} = \dfrac{193,000}{15,740} = 12.2$ lbs. sq. in. load on net area of piers.

Then the load on a narrow side corner would be,

$$W = 1215 \times 12.2 = 14,800 \text{ lbs.}$$
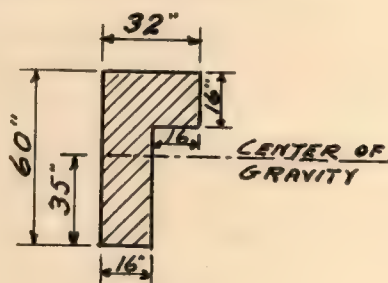
Fig. 6

and for I (Fig. 6)

$$I = \frac{(16 \times 35^3) + (32 \times 25^3) - (16 \times 9^3)}{3} = 391,400$$

$$\frac{I}{C} = \frac{391,400}{35}$$

Then,

$$S = \frac{Wlc}{2I} = \frac{14,800 \times 60 \times 35}{2 \times 391,400} = 39.7 \text{ lbs. per sq. in.}$$

Thus the stress would be only 39.7 lbs. per sq. in., in bending tension, which is well under the ultimate, and failure would not be from bending. This is the
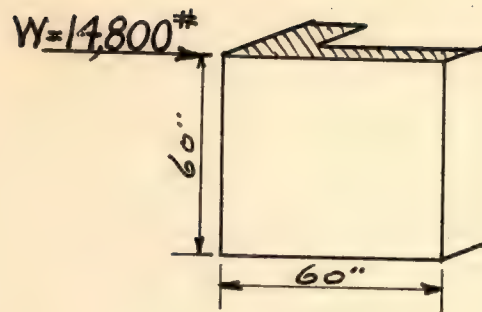
Fig. 7

most critical point in the building and should be safeguarded by keeping the piers as wide as possible on the narrow side or by increasing the thickness of the masonry if the piers are narrow, or by reinforcing steel rods built into the piers.

The above analysis based upon an actual building of the usual proportions. Modifications of this design would lead to greater or less stability and the latter might, under certain conditions, approach the dangerous from an earthquake standpoint, for example, if the end piers had been 30 inches wide instead of 60 inches with the percentage of openings remaining the same, then the unit load would have also remained the same, but the bending stress would have been increased as shown by the following:
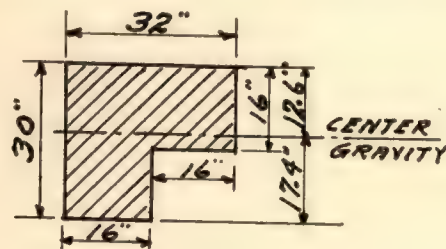
Fig. 8

$$I = \frac{(32 \times 12.6^3) + (16 \times 3.4^3) + (16 \times 17.4^3)}{3} = 49,700$$

$$\frac{I}{c} = \frac{49,700}{17.4}$$

$$W = A \times w = 737 \times 12.0 = 9,000 \text{ lbs.}$$

Similarly then,

$$S = \frac{Wlc}{2I} = \frac{9,000 \times 60 \times 17.4}{2 \times 49,700} = 94.2 \text{ lbs. per sq. in. bending stress.}$$

A bending stress of 94.2 lbs. per sq. in., while not necessarily hazardous, does not, on the other hand, provide a very great margin of safety against an earthquake of the intensity herein assumed, and it would seem advisable in case such a condition is unavoidable, to look into the matter of providing some additional security for these critical corners of the building. In this particular case reinforcing bars

could be used to relieve to any desired degree the tensile stresses induced in these piers, involving a minimum expenditure and providing a maximum effectiveness.

Observations

It has been shown that the average building assumed here would not suffer damage during an earthquake of one tenth gravity intensity.

Brick masonry is primarily a compression material and as such it resists compression on either a vertical or diagonal line, its actual value being measured by the strength of the mortar up to a certain point and beyond this point by the strength of the brick.

The question of "diagonal tension" in masonry is quickly answered when properly understood. The term "diagonal tension" is a technical name given to a somewhat unknown stress developed in a reinforced concrete beam acting from the top of the beam at the support diagonally toward the bottom of the beam. Hence there is no technical "diagonal tension" in a brick wall or pier.

There may be, however, tension or compression set up in a diagonal direction.

It was shown that in the individual piers in the first story of the building analyzed, the shearing stresses induced, were far below the shearing strength of the masonry. It was also shown that the secondary bending stresses in the piers were higher than the shearing stresses and the ones to be more considered.

It is perhaps the failure in *bending* and not in *shear* that accounts for the diagonal cracks which have appeared in some brick piers after a heavy quake.

It is readily seen that the more slender the pier in proportion to its height the greater the bending stress will be and vice versa, a wider or heavier pier reduces the bending stress. This is one of the critical points to be studied in proportioning and grouping windows. Slender pilaster piers between windows are useless for lateral resistance, while, if the windows are grouped together and the extra pier masonry is added to the masonry mass on either side of the window group, then the resistance would be materially increased. This is simple mathematics and common sense, but a very important element in the lateral resistance of a building.

It must be apparent to anyone who has read this article that to condemn a brick bearing-wall building as being any more hazardous than a building of any other kind of *material*, is folly. The principles set forth here as applying to brick bearing-walls are identically applicable to walls of concrete, granite, or even cast iron of the same relative compression and shearing strength. None of them will "bend" very much.

Suggestions for Added Security

Many architects and builders are incorporating added measures of security in their structures, having particularly in mind the matter of lateral resistance. These are simple results of the considerations shown above.

Pipe chases, etc., should not be permitted in the critical piers or in walls in the narrow dimension of a structure, where walls are of minimum thickness.

Parapet walls, chimneys, etc., should be laid up in cement-mortar from the top to a distance below the roof line.

When piers are narrow on the narrow sides of the structure, a vertical reinforcing steel rod on each side of the pier will take up the bending tension.

Reinforcing bars laid in the walls under floor and ceiling joists, trusses, beams, etc., and running continuously around the building, provide a more uniform distribution of the loads and also aid in tying together, hence are desirable in certain conditions. Reinforcing bars laid in the masonry are superior to a reinforced concrete band (which is better than no band), inasmuch as there is no cleavage joint between two different materials, and the brick having a better adhesive surface for mortar than the concrete.

Girders and beams should be provided with means to distribute an end thrust into the wall so as not to "punch" the wall.

The mortar for face brick and the common brick backing should be of the same strength to allow the whole thickness of the wall to act as a unit.

Finally

Finally, it should be particularly remembered that where experienced certificated architects supervise the work, or where experienced, competent builders are engaged, or where good workmanship is obtained through enforcement of adequate building codes, or where all of these conditions prevail as they generally do in the larger cities, the *solid brick* bearing wall building is a type of construction which needs cause its owner no anxiety for its adequacy to resist seismic disturbances.

The Hollow Masonry Unit in Earthquake Resistant Design

Properties of Hollow Clay Tile

Perhaps no material has been the basis of so much misconception in connection with earthquake damage as hollow clay tile. It appears that much of this misconception has been due to the fact that the construction industry generally has not understood correctly the uses of different kinds of tile and that much of it has been used for purposes other than those for which it was intended. Some explanation of the characteristics of clay tile is then desirable.

The earliest extensive use of clay tile was for partitions with similar development as a material for fireproofing, furring, etc. Partition tile is not designed for load bearing purposes beyond a proper margin of safety in carrying its own weight.

Because of the many admirable features of hollow clay tile, particularly its lightness, rapidity of construction and convenience for plastering, it was adopted for exterior walls. The manufacturers of the product, however, realized that in such conditions a different type of product was desirable and hence have developed what is known as the "load bearing," sometimes called "exterior" tile, made with thicker walls, more numerous interior webs and manufactured to confirm to rigid specifications promulgated by the American Society for Testing Materials.

These specifications include the following requirements for medium load bearing wall tile. (A. S. T. M. C34-27.)

	Mean of 5 Tests	Individual
Absorption (maximum)	16%	19%
Compressive strength per sq. in. gross area—		
End construction (minimum)	1400 lbs.	1000 lbs.
Side construction	700 lbs.	500 lbs.

There are a considerable number of manufacturers of load bearing clay tile in California and they are all willing to guarantee their tile to comply with the A. S. T. M. standard. There is a very wide variety of sizes, shapes and designs, each adaptable to some special use and this very variety has increased the confusion which has been referred to above.

In line with the tendency so strongly evident in the clay products industry to base all recommendations upon the most accurate technical data, the Bureau of Standards at Washington was requested to make a series of tests on full sized masonry walls constructed of hollow tile. Some of the important results of this series of studies made in 1925-1927 may be briefly indicated.

1. The quality of mortar used was of vital importance in determining the strength of the resulting wall. Lime mortar proved inadequate even with the best tile, developing a strength of less than one-half or in many cases less than one-fourth that obtained by the use of lime and cement mortar. Straight cement mortar gave no higher, and in some cases lower, strength than lime and cement, the strongest wall resulting from the mixture that contained the least amount of sand relative to the cementing materials. In other words, a much better wall resulted from the use of a mortar one part cement, one and one-quarter parts of lime and four of sand than from a mortar one part of cement, one and one-quarter lime and 6 of sand.

2. With walls constructed with tile qualfying as load bearing tile by the A. S. T. M. or Government standards, and laid up with lime cement or straight cement mortar, the minimum average compressive strength was 275 pounds per square inch gross wall section, while the maximum reached 610 pounds per square inch. The average of these walls was in excess of 400 pounds. The individual tile with which these tests were made had an average compressive strength per square inch of gross area of about 1500 pounds when tested on end and about 550 pounds tested on the side. (A recent series of tests on load-bearing tile from Northern and Southern California has shown values in excess of 1600 pounds per sq. in., tested on the side.) The variety of size and shape of the tiles was so great, however, that the authors of the paper report that the ratios of wall strength to tile strength appear to cover so wide a range that they have no useful meaning.

3. The strength of mixed construction walls where the tiles were laid alternately on the side and on end was distinctly lower than either straight end construction or straight side construction walls.

4. Except where laid up with very superior workmanship the side construction wall gave a higher strength in compression than end construction and on transverse strength tests the side construction wall generally appeared stronger than the end construction. The comparison would probably have been even more favorable to side construction had it been possible to make the comparison with tile designed for use in side construction as the above comparisons were all based on tile designed for end construction, but tested also on the side.

5. The resistance of the 8 inch walls to side pressure was equivalent to from 41 to 115 pounds uniform loading per square foot, while the 12 inch walls resisted side pressure of 120 to 150 pounds per square foot. Comparing end construction with side construction using a similar series of mortars and identical tile, the end construction showed an extreme fibre stress under transverse load of 41 and 53 pounds per square inch, while the side construction tiles gave values of 62 to 98 pounds per square inch.

It may be interesting to refer these results to those cited in Chapter VIII for brick masonry and also obtained by the Bureau of Standards. It will be seen from such a comparison that in compression the brick wall had about 600 pounds per square inch, while the hollow tile wall had about 400 pounds per square inch, or some 66% of the brick.

In the design of buildings, however, one of the most important factors is the dead weight of the material itself and here hollow tile has a substantial advantage. A 12x12x12 hollow tile conforming to the A. S. T. M. specifications weighs about 48 pounds, whereas a cubic foot of brick masonry weighs about 120 pounds. In compression an 8 inch brick wall is substantially equivalent to a 12 inch tile wall, the 8 inches of brick weighing about 80 pounds and the 12 inches of tile about 50 pounds.

With regard to the resistance in shear of hollow tile construction it may be pointed out that the edges of load bearing tile are sharply scored, which permits an excellent bond between the mortar and the tile. Mr. W. M. Butts, who has made some study of this subject, in a report to the Clay Products Institute, states: "If the most critical angle of failure is chosen the calculated value of the hollow tile wall in shear will be about 125 pounds per square inch of gross section. These values are for failure under ultimate load. It seems reasonable to assume that the proportion of the crushing values will not be lower for tile than for brick work as the former is made under generally superior conditions and under greater pressure. A value in shear of 50 pounds per square inch of gross section seems decidedly conservative and if the tile snugly fills the opening for the filler wall it will assuredly offer resistance of at least this amount."

Hollow Tile for Filler Walls

Much of the discussion in Chapter VIII respecting the advantages of unit masonry in filler wall construction applies equally to the use of hollow clay tile. Load bearing tile rather than partition tile should always be employed for filler wall purposes both because of the desirability of bracing the structure and of offering the requisite fire resistance. There is absolutely no reason in the material itself why a filler wall of hollow clay tile cannot be made to resist any desired earthquake stress. In practise, however, if the building is designed in such a way that the walls are expected to provide a large proportion of the rigidity, it will be found in certain conditions

that an excess thickness of wall would be required and hence the use of hollow tile might be uneconomical as compared with other material. It is strictly a matter of engineering design.

By reference to the analysis presented by Mr. Butts in Chapter VII, it will be observed that in his assumed structure the shear per square inch of wall section ran from 10.1 pounds in the twelfth story up to 182 pounds in the first story, all of the stress being taken up by the walls and none of it by the frame. Even in this condition hollow tile would be entirely adequate for the upper 5 stories of this building, assuming only a 12 inch wall and without making allowance for the reduced dead load due to lighter walls.

But as we have seen it is frequently more economical to design the building so that the steel or skeleton frame will supply the greater part of the resistance to the lateral forces. In this case, as in the case of a similar brick wall, the wall itself is not counted on to supply the resistance and its main function is to remain intact in place. The 12 inch tile wall then has a notable advantage over either brick or concrete in that it is so much lighter that it materially reduces the dead weight on the structure. Whether it is used or not will depend on other considerations than earthquake danger.

There is little danger of the tile, if laid up with reasonable workmanship and proper mortar, being thrown out of the wall but its own weight under an earthquake shock. Again quoting from Mr. Butts, in the report referred to above and bearing in mind the transverse strengths of hollow tile walls established by the Bureau of Standards in the report above quoted, "as a maximum load which an earthquake of an intensity of 1/10 g. would set up is about 7 pounds per square foot, which would induce an extreme fiber stress of 8 pounds per square inch. Even the poorest materials and below average workmanship should be safe."

It is, of course, highly important that tile walls should be laid up snugly against the spandrel beams and supporting columns if it is to be counted on to offer any resistance. This need not be discussed in more detail as the explanation with respect to brick filler walls applies fully to hollow tile.

In the chapter describing the results of the Santa Barbara earthquake the experience of several skeleton frame buildings with tile filler walls was cited. Load bearing tile had not been developed at the time of the San Francisco earthquake and was not in use in Japan so that it is not possible to cite more extensive history of this material in actual performance under earthquake conditions.

The use of hollow tile in filler wall construction is a matter of engineering design. It is undoubtedly one of the best understood materials due to the extensive studies made by the Bureau of Standards and others, has the great advantage of lightness and if used within its limitations of strength is entirely dependable.

Hollow Tile in Bearing Wall Buildings

From what has been said above it should be apparent that, with the strength values in compression and under transverse pressure definitely established on the highest authority, the designer of bearing wall buildings employing load bearing clay tile can safely build such structures with every assurance of their being satisfactory from an earthquake resistant standpoint.

In practise due to considerations of economy of floor space or other requirements, hollow clay tile is rarely used for bearing walls for buildings exceeding two stories. In a two story building it is readily seen that the chief loads are on the second floor and the roof. Allowing the customary 50 pounds live load for floors when brought to the walls, 20 pounds live load for roofs and with 25 pound dead load on each floor and also allowing 50% openings, the direct load on the wall in bearing is only 11 pounds per square inch, or far inside of the minimum strength of such wall.

A residence of 1500 square feet ground area, of 2 stories, puts a stress in shear during an earthquake of 1/10 g., of about 280 pounds per square foot on the wall, or about 2 pounds per square inch. As shown above, this is far within the danger limit. Even with a very large structure there is little likelihood of failure in shear or bending.

As in the case of the brick bearing wall structure, the critical factor is generally the moving of the interior frame. Precaution should be taken to anchor thoroughly the floor joists to the bearing walls and this can readily be done by embedding the anchors into the cells of the tile. Pitched roofs should be strongly fastened to the ceiling joists to form a truss so as to obtain vertical bearing on the walls without side thrust.

Hollow tile has a special advantage in that lintels may be assembled, reinforced and concreted on the ground as a "loose lintel" and placed in position after being set, thus providing a wall of homogeneous material.

As in the case of other masonry, the quality and use of mortar is of vital importance. Horizontal and vertical joints should be filled and in the case of side construction the ends of the shells and webs should be well covered with mortar.

What has been said as to the engineering characteristics of hollow tile is well borne out by experience. It has been previously pointed out that there were no failures of load bearing walls constructed of hollow building tile in Santa Barbara, although there were some cases where the roof joists improperly anchored threw down portions of the parapet walls. In other earthquakes the experience has generally been equally satisfactory, the exceptions being where flagrant violations of basic principles of design were committed.

In view of these facts it is hard to see how any prejudice can exist against hollow tile buildings for earthquake construction when used within the proper limits of their determined strength and with a regard to the fundamentals of building construction which have been indicated.

Hollow Tile As Partition Material

Partitions are not expected normally to provide any bracing for buildings to resist earthquake stress, nor to carry any load beyond their own weight. Their chief function from an earthquake standpoint is to remain in place and in case of the frame being distorted. to remain intact, Load bearing tile is therefore not employed for this purpose and partition or interior tile is used. The signficant feature in the experience with hollow tile partitions is that *no hollow tile set in cement-lime or cement-mortar and properly plastered have ever yet been shaken out of partitions in Class A buildings.*

In cases where cracks have developed through the distortion of the entire frame the repair cost is reduced to a minimum with clay tile. In comparing clay tile partitions with metal lath and plaster, which is a common competitor, it is important to bear in mind that the metal construction will distort with the structure, and having no compression strength, will stretch or twist or in many cases buckle and cause diagonal and other cracks through the wall. It is difficult, if not impossible, to properly repair a metal stud after it has once distorted, as the kinks or bends have to be straightened out before the new plastering can be done properly. A buckled or bent partition must be taken down completely. In either case the cost of repair is excessive.

With clay hollow tile construction the mortar joints around the hollow panel at the walls and ceiling have a chance to yield to some extent before the wall is affected and even if the wall is affected it has the other mortar joints to work upon. When a tile wall is cracked by a quake it is very quickly and perfectly repaired with mortar at very slight cost, as only the crack itself needs attention.

The engineering commission appointed by the United States Government after the San Francisco disaster reported that nearly all the hollow tile walls in such buildings as the Flood, Call, Chronicle, Post Office, Fairmont Hotel, etc., were standing up well and such tile partitions as failed were reported as being laid up in "lime mortar which crumbled in the hands and were not in Class A buildings."

Likewise in Santa Barbara the well built hollow tile partitions including those in buildings with hollow tile bearing walls were undamaged.

A caution should be entered here against a practise that has occasionally been too common, of laying nailing strips in the horizontal joints of the tile walls. The serious result of this practise was evident in the partitions of the St. Francis Hospital, Santa Barbara, where full sized wood lath were laid in every third course of the partitions. Many of these partitions were badly damaged, in each case the failure occurring along the line of these lath. Of course

mortar cannot be expected to hold when separated by wood from the masonry to which it is supposed to adhere. If nailing strips are employed they should be used in the vertical joints and sparingly, but in no case between the ends of the tile, as this involves too large a reduction in the effective mortar.

Hollow Units of Other Material

From the discussion of the characteristics of hollow clay tile the question naturally follows as to the performance of hollow concrete block. The problem is not merely one of the strength of the individual unit. The highest grade of concrete block has a strength on end of about 1000 pounds per square inch of gross sectional area, as compared with 1400 pounds required by A. S. T. M. specifications for medium load bearing clay tile. Concrete block is not recommended for use in side construction and therefore must be employed with the reduced mortar bed permitted by the webs themselves. There are no records of public or authoritative tests on concrete block *masonry* so far as we are aware. The studies on both brick walls and those of hollow clay tile, however, demonstrate that the wall strength is closely related to the fullness of the mortar bed. The mortar bed in concrete block masonry can never exceed 70% of the full area of the wall and in general is less than 50%.

But even more important than the area to which the mortar may adhere is the *bond* that can be obtained. As discussed under the question of "Reinforced Concrete" (page 86) it is extremely difficult to bond a cement mortar to concrete. This was demonstrated by Washington State University tests which showed the adhesion of mortar to the burnt clay product more than twice that obtained to concrete using the same mortar.

Experience with the concrete blocks amply demon-

86a—Collapse of a concrete block building, due to earthquake.

No. 86—Wreck of concrete block building in Santa Barbara after the earthquake showing lack of mortar adhesion.

strates the importance of this element. Photograph No. 86, taken after the earthquake at Santa Barbara shows a hollow concrete block building where the walls gave way. It will be noted that the individual blocks are in good condition but that, as observed on the clean joint at the side, the mortar did not adhere to the tile. The photograph shows that the concrete block in question were so called "Grade A" block conforming to the Underwriter's specifications as far as can be observed.

A considerable amount of this type of construction has been employed in Florida and suffered disastrous results when subjected to the hurricane pressure in both Miami and the Palm Beach storms. In an article in the Engineering News-Record for October 4th, 1928, describing the latter disaster, R. S. Tilden brings out this record clearly: "The buildings of concrete block or frame covered with corrugated metal (in this district), are in many cases a complete loss and almost all are seriously damaged. . . . The warehouse of the Southern Storage Company, a new building of concrete block, is a complete loss, as the illustration shows." The detailed photographs clearly bring out that the cause of the damage was the failure of the mortar to adhere properly to the block. While there were some failures of hollow clay tile buildings in the same disaster, these resulted from the use of the wrong material, as pointed out by Mr. Tilden, who states: "It is significant that the hollow tile as used in East Palm Beach and vicinity does not have the thicker web and shell of what is commonly termed 'load bearing,' tile but the thinner section of partition block."

Both the technology and the history of the concrete block would thus indicate that it is of limited utility from the standpoint of earthquake resistance because of its inadequate bond with the mortar and the fact that it must be laid on end construction which so greatly reduces the effective mortar bed.

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Recent Tendencies of Building Construction

The selection of the materials to be used in the construction of any building is of course, dependent upon many factors. We have been discussing in the earlier chapters the factor of resistance to earthquake stress.

In the preceding chapters the evidence and discussion has, it would seem, established the following propositions:

1. The monetary loss to building construction which has been caused directly by earthquake shocks has been greatly exaggerated. In the three outstanding disasters which have been studied the loss in the seriously affected area was only from 5 to 10 per cent of the total value of the buildings in that area.

2. Buildings constructed of brick, tile and terra cotta have, on the whole, given remarkable records of stability in earthquakes. What is commonly known as "good construction" has a margin of safety with respect to the action of earthquake forces and has satisfactorily withstood the additional strain at times of seismic disturbance.

3. The proven strength of brick and load-bearing hollow tile masonry in compression, bending and shear demonstrates that in designing structures with special reference to resisting earthquake forces, these clay products are both reliable and economical, both in bearing wall and skeleton frame structures.

4. The failures in buildings constructed with clay products, relatively few in number, have been due to a disregard of the proper use of the material, to location directly on a fault line or on unstable soil or to violation of fundamental construction principles.

5. Other construction materials proposed as a substitute for clay products must be judged by their actions under actual rather than ideal conditions. Especially the effects of time and the elements in the disintegration or weakening of the materials must be borne in mind with reference to their earthquake resistance.

That the earthquake question is but one, and a comparatively minor, consideration is self evident. Advantages of permanence, fire resistance, cost, resistance to the action of heat, cold, wind, rain and sound

No. 87—Chanin Building, 62nd Street, New York, Sloan and Robertson, architects. Chanin Brothers, contractors. Face brick and terra cotta exterior, brick filler walls, clay tile partitions.

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No. 88—A portion of the Medical Center group, 168th Street and Riverside Drive, New York, James Gamble Rogers, architect. Mark Eidlitz & Sons, contractors. Face brick exterior, hollow tile backing, interior partitions of clay tile. Photo courtesy Architectural Forum, N. Y.

penetration, and not least the element of beauty, all support the selection of the appropriate burnt clay material. It lies beyond the scope of this book to discuss these elements in detail and indeed, they are so well recognized by architects as to need little, if any, proof. That the leading architects throughout the country design their outstanding buildings of these materials is sufficient evidence of their acceptance of this fact.

Some examples of recent construction of this type may be given.

No more striking building has been built of recent years than the Chanin Building on 42nd St. and Lexington Ave., New York. This structure of 56 stories, designed by Sloan and Robertson, architects, has face brick exterior with terra cotta trim. The backing of the walls is of common brick and all important partitions are of clay tile. Photograph No. 87.

Towering above the Hudson River at 168th Street, New York City, rises the magnificent new Medical Center group. No picture yet taken can give an adequate idea of the immensity and beauty of this construction. The architect, James Gamble Rogers of New York, selected face brick for the exterior with 8 inch load bearing hollow tile backing and a 3 inch tile furring. The interior partitions in accordance with good hospital practise are all of clay tile. The work thus far completed has cost over $20,000,000 exclusive of site and an additional amount nearly as great is to be expended in the future. Mark Eidlitz and Sons are the contractors. A photograph of a small portion of the group is shown as No. 88.

No. 89—New York Central Building, Park Avenue, New York. Warren and Wetmore, architects. James Stewart & Co., contractors. Face brick and terra cotta exterior, common brick and hollow tile backing. Fireproofing chiefly of tile and brick. Important partitions are clay tile.

Standing right athwart Park Avenue rises the magnificent New York Central Building for which Warren & Wetmore were the architects. Photograph No. 89. It covers 3 blocks of ground area and its gilded roof is one of the outstanding features of the uptown New York landscape. Its chief walls are of face brick with common brick and hollow tile backing. All important partitions are of clay tile and the columns are

No. 90—Wardman Park Hotel, Washington, D. C., before construction of new addition. Frank R. White, architect of original structure. M. Mesrobian, architect of addition. A bearing wall structure of common brick and face brick. Partitions of new addition chiefly clay tile. (See text).

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No. 91—*Union Trust Building, Detroit. Smith, Hinch-man and Grylls, architects. Showing adaptability of brick and terra cotta to modernistic design. Face brick and terra cotta exterior, common brick backing walls, important partitions of hollow clay tile.*

No. 92—*Carbide and Carbon Building, Michigan Avenue, Chicago. Burnham Brothers, architects. Paschen Brothers, contractors. Terra cotta and face brick exterior, common brick backing with clay tile partitions.*

chiefly fireproofed with clay hollow tile.

The nation's capital offers many examples of outstanding construction. The newer Government buildings all exemplify the use of clay products for both architectural and structural uses. Instead of presenting, however, a Government building we should like to call particular attention to the Wardman Park Hotel, residence of many nationally known statesmen. Frank R. White was the architect of the original structure shown in photograph No. 90. There is now

in progress a new addition containing 265 rooms of which M. Mesrobian is architect. This structure is a bearing wall building of common and face brick. It is interesting to know that while in the original building only a limited amount of hollow tile partitions were used, in the new addition this type of partition material predominates.

The adaptability of clay products to modern design is one of their notable advantages. The new Union Trust Building in Detroit designed by Archi-

No. 93—Stevens Hotel, Chicago. Largest Hotel in the World. Holabird and Roche, now Holabird and Root, architects. Geo. A. Fuller Co., contractors. Exterior walls, face brick backed by common brick. Interior partitions of hollow clay tile.

tects Smith, Hinchman and Grylls exemplified this feature. Photograph No. 91. The exterior walls are face brick, with common brick backing. Elaborate decorations in terra cotta are found at the 6th and 7th floor levels and also ornamenting the upper portions of the structure. The chief partition material is clay hollow tile.

The name of D. H. Burnham is widely known for the best in architecture. The successor firm, Burnham Brothers of Chicago, is now completing the Carbide and Carbon Building, Michigan Avenue, Chicago, a 43 story structure of which Paschen Brothers are the contractors. Photograph No. 92. This building is distinguished by a dark green terra cotta exterior which is matched on the property line walls with green face brick. Throughout the building all partitions are of clay tile.

The largest hotel in the world is said to be the Stevens Hotel, Michigan Avenue, Chicago, designed by Holabird and Roche, now Holabird and Root, architects. Photograph 93. This enormous building is constructed with face brick exterior with common brick backing walls. All of the interior partitions are constructed with clay tile, so as to give soundproofness as well as other features so important in hotel construction.

Of late years theaters have become one of the outstanding types of buildings throughout the country. The most recent notable structure of this kind is the Mastbaum motion picture theatre at Philadelphia, said to be the second largest building of this kind in the world. Hoffman and Heenan, Philadelphia, architects, designed the building which is of light colored face brick beautifully ornamented with terra cotta. The partitions are clay tile. This building was opened on February 28th, 1929 and its beauty and appropri-

No. 95—Dade County Court House and Miami City Hall,
Miami, Florida. A. Ten Eyck Brown, architect. August
Geiger, associate. L. W. Hancock, contractor. All terra
cotta exterior, load bearing hollow tile backing, clay tile
partitions.

No. 94—Bankers Building, Chicago. Completed October,
1927. Architects Burnham Bros., Chicago. Contractor,
Dilks Construction Company, Chicago. An imposing
structure of modern monumental style. Face brick exterior
with terra cotta ornamentation, common brick backing, clay
tile partitions.

ateness for the purposes intended have attracted na-
tionwide attention.

Florida has been constructing some notable build-
ings in the last few years. At Miami has just been
completed the Dade County Court House and Miami
City Hall. A. Ten Eyck Brown was the architect
with August Geiger as his associate. The contractor
was L. W. Hancock. This beautiful building shown
in photograph No. 95, is entirely faced with beautiful
white terra cotta. The exterior walls are backed with
load bearing clay tile and the partitions are exclusively
of clay tile.

In the older sections of the country builders have
learned that it is economical to erect structures that
will endure. Many are the substantial homes of brick
that are more than 200 years old. Even on housing
projects of a more commercial nature the permanence,
low cost of upkeep and the fire resistant character of
brick has led to its very general adoption. As an ex-

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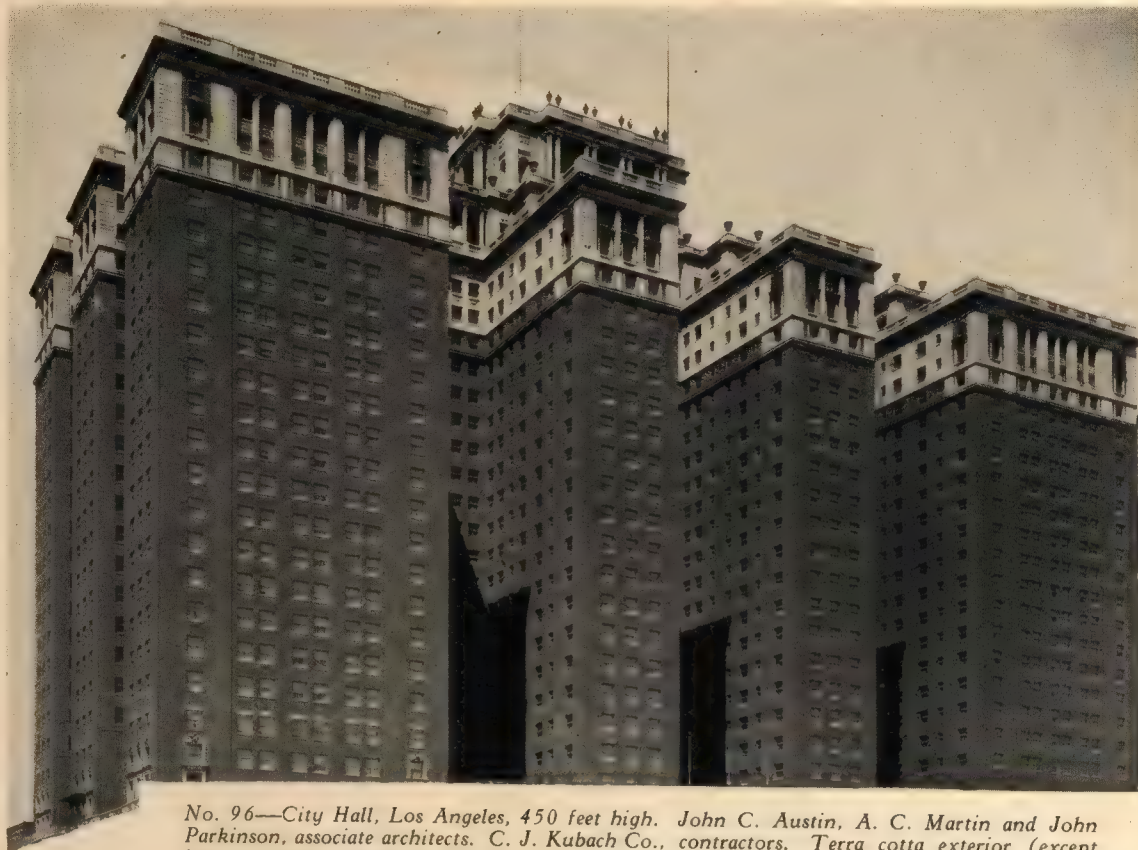
No. 96—City Hall, Los Angeles, 450 feet high. John C. Austin, A. C. Martin and John Parkinson, associate architects. C. J. Kubach Co., contractors. Terra cotta exterior (except lower stories), enameled brick on light courts, all backing of common brick. Clay tile partitions. Liberal use of interior ornamental tile and acoustic clay tile decorations, also large areas of paving brick promenades. Wings have clay tile roofs.

ample of this sort of building project, a permit was taken out in Baltimore on February 11th for 120 two story common brick houses to be erected by the Donohue Home Building Company in East Baltimore.

But it is not necessary to go to the East to find worthy examples of clay products construction. The leading architects of the Pacific Coast are fully appreciative of the advantages indicated. Only a few buildings of varying uses can be given because of the limitations of space. One of the most outstanding buildings in Southern California is the recently completed Los Angeles City Hall. John C. Austin, A. C. Martin and John Parkinson, the architects associated in this building shown in photograph No. 96, selected architectural terra cotta as the principal facing material and specially finished and matched enamelled face brick for the large interior light courts, both of which together with the granite of the lower stories, were backed with brick walls. All partitions are of clay hollow tile and the wings are covered with clay roof tile.

Government construction in California has on the whole been notably good both structurally and artistically. On the earlier pages the splendid records of the Government buildings in San Francisco during the disaster of 1906 and of the Post Office at Santa Barbara in 1925 have been detailed. The James W.

Wadsworth Hospital at the National Home for Disabled Volunteer Soldiers, Sawtelle, is a splendid example of face brick construction combined with the use of clay roof tile. Interior partitions are hollow clay tile in accord with good practice in hospital construction. A picture of the hospital which was designed by the Treasury Department architect at Washington and built by the Los Angeles Contracting Company, appears as No. 97.

Unsurpassed as an example of modernized romanesque architecture is the group of buildings for the University of California at Los Angeles, designed by Allison and Allison, architects; George W. Kelham, supervising architect; H. J. Brunnier, structural engineer, and constructed by Bannister & Gow, general contractors, and by Pozzo Construction Company, general contractors; photograph No. 98.

The color and texture of the designs are expressed in face brick with terra cotta, while all backing is of common brick. This together with the use of clay tile roofing, clay hollow tile for partitions and the generous use of decorative tile and face-brick work in the interiors gives the buildings combined beauty and permanence in the highest degree.

San Francisco and Oakland have contributed some of the most notable and beautiful examples of modern

No. 97—James W. Wadsworth Hospital, National Home for Disabled Volunteer Soldiers, Sawtelle, California. Supervising Architect, Treasury Department Architect. Los Angeles Contracting Company, builders. A fine example of Government construction using face brick, roof tile and clay tile partitions. This same general type has been adopted for further units in the building program at this home.

No. 98—Josiah Royce Hall, one of the buildings in the University of California group at Los Angeles, Allison and Allison, architects. Geo. W. Kelham, supervising architect. H. J. Brunnier, structural engineer. Bannister & Gow, contractors. Face brick and terra cotta exterior, clay tile partitions, brick and tile interior decorative work. Clay tile roofs. A beautiful example of Italian Romanesque.

No. 99—A notable group of clay products structures on Montgomery Street, San Francisco. In the center is the Russ Building, 32 stories. Geo. W. Kelham, architect. H. J. Brunnier, structural engineer. Dinwiddie Construction Co., contractors. Terra cotta and face brick exterior with brick backing walls, clay tile partitions. Next to left is the Alexander Building, 16 stories. Lewis P. Hobart, architect; T. Ronneberg, structural engineer. Face brick with brick backing walls and clay tile partitions. At extreme left is the Hunter-Dulin Building, 22 stories. Schultze and Weaver, architects; H. J. Brunnier, engineer. Lindgren and Swinnerton, contractors. Face brick and terra cotta exterior with brick backing walls, clay tile partitions, clay tile roofings. At extreme right, California Commercial Union Building, 16 stories. Geo. W. Kelham, architect; H. J. Brunnier, engineer. Face brick and terra cotta exterior, common brick backing, clay tile partitions.

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No. 100—Small portion of residence of Mr. O. Nicholas Gabriel, San Marino, California. Roland E. Coate, architect, Los Angeles. A beautiful true "California Style" solid brick residence employing also clay roof tile.

No. 101—Residence of Mr. J. W. Green, Flintridge, California. Paul R. Williams, architect. Brick veneer on wood frame. Notice the skillful use of brick for decorative work and chimney construction.

No. 102—Courtyard in Arcade Building, Pasadena. Marston, Van Pelt and Maybury, Architects. A beautiful example of the use of common brick in a commercial building, the California style being fully carried out by the use of clay roof tile.

No. 103—Residence of Mr. H. F. Haldeman, Beverly Hills, California. Marston, Van Pelt & Maybury, architects. Pasadena. Brick veneer on wood frame. A restful and appropriate treatment of brick residence construction.

building design and construction executed in recent years. In these cities the engineers, because of the memory of 1906, have given very definite thought to the matter of earthquake resistant design and at the same time have been desirous of obtaining the greatest possible beauty and utility with economy. They have found clay products admirably adapted to their requirements. Photograph No. 99 shows a section of Montgomery Street in which are found four notable structures. The central portion of the photograph shows the Russ Building, a steel frame structure rising with appropriate set backs, 33 stories, having a terra cotta and face brick exterior with brick backing. Partitions are clay tile. George W. Kelham was the architect, H. J. Brunnier consulting structural engineer and Dinwiddie Construction Company the general contractors.

In the same photograph the next building to the left is the Alexander Building, Lewis P. Hobart, architect; T. Ronneberg, consulting structural engineer. This building, 16 stories high, has an exterior of face brick with brick backing walls and all partitions are of clay tile.

At the extreme left is shown the upper portion of the Hunter-Dulin Building. With a structural steel frame this building has a terra cotta exterior with brick backing walls and clay tile partitions. An interesting use of clay roof tile is shown. Schultze and Weaver of New York and San Francisco were the architects; H. J. Brunnier, engineer, with Lindgren and Swinnerton, Inc., as general contractors.

At the extreme right of the photograph is shown the California Commercial Union Building, also designed by Geo. W. Kelham, H. J. Brunnier, engineer. The exterior of this building is of library gray face brick and terra cotta with common brick backing. Clay tile was used for partitions.

Through many years of successful use both common and face brick have established their position as the most beautiful as well as the most enduring materials for residence construction. Homes in the English, French or Colonial style naturally call for the use of these materials. The tendency on the part of leading architects to develop the newer Mediterranean or California style of architecture, has called upon these old materials to take newer forms, a new softness of texture and beauty and still retain their natural honest appearance of structural strength, their fire and weather resistance and enduring comfort. Photographs of three typical homes recently constructed by well known architects showing various uses of the materials are found in numbers 100, 101, and 103.

Indicative of the same possibilities in the use of brick is its employment for commercial buildings. Almost universally used for larger structures, brick and tile in the hands of a skilled designer lend themselves also to the smaller structures as exemplified in photograph No. 102.

Only the limitations of space prevent the use of many other illustrations to show the manner in which the leading architects, contractors and builders throughout California have successfully employed clay products in solving their construction problems. One fact at least should not be overlooked. This is the policy of 3 of the oldest and largest California corporations which have had the opportunity to experience whatever earthquakes have occurred in the state and also have large engineering staffs competent to study all the facts or theories which may have a bearing upon the solution of their building problems. The Southern Pacific Company, Standard Oil Company of California, and Pacific Telephone and Telegraph Company, all have their main offices in San Francisco. All have constructed many buildings since the San Francisco fire and earthquake and since the Santa Barbara earthquake. Practically without exception all of these structures have used brick as their basic material. The Southern Pacific Company not only has one of the outstanding office buildings in San Francisco but has built many stations and warehouses throughout the state. The Standard Oil Company has notable office building structures in both San Francisco and Los Angeles which employ common and face brick and terra cotta, while the Telephone Company observing the resistance to earthquake and fire of their old building on Bush Street, San Francisco, (See photograph No. 11a) not only constructed one of the most beautiful buildings in California on New Montgomery Street, San Francisco, employing a structural steel frame with brick and terra cotta walls and clay tile partitions, but have built many other buildings for offices and exchanges in all parts of the state with these materials.

Thus we find in California as elsewhere throughout the United States, a recognition of the leadership of burnt clay products. Those architects who are leading the way in their profession find that their ideas can be most satisfactorily developed and perpetuated in using materials of proven strength, having permanency of color and texture. Those contractors who take pride in their work and who know their costs prefer to employ this well tried adaptable material. Those owners who through careful study put themselves in possession of the facts find that their requirements will be best served, their investment protected and their cost of upkeep minimized by the use of clay products.

The oldest building material in the world, developed and standardized by modern skill maintains its preeminence through the trials imposed by nature and the tests applied by man.

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